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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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THE SHANDAKEN TUNNEL

By R. W. Gausmann,* M. Am. Soc. C. E.

To Be Presented June 1, 1927

Synopsis

The Shandaken Tunnel extending between Prattsville and Shandaken (mostly in Greene County), N. Y., is about 18 miles in length, the longest yet built. It connects two water-sheds, making the run-off of Schoharie Creek as impounded by the Gilboa Dam on the north available to the Esopus water-shed on the south, already serving the City of New York.

The paper recounts the development of this project and the geology of the region. The actual construction work is covered in detail, both construction shafts and tunnel proper. Drilling, dynamiting, mucking, timbering, and concrete lining operations are described; also mechanical equipment, progress, and costs.

Some special features, such as the use of the concrete gun, organization problems for efficient progress, and transportation, are treated incidentally. Actually, the construction was under way from November, 1917, to October, 1924.

General

The Shandaken Tunnel forms a part of the Catskill Water Supply System of New York City. Its function is to carry water from Schoharie Creek to Esopus Creek, from which the flow is to the Ashokan Reservoir (Fig. 1). The tunnel is 18.1 miles long, being the longest continuous tunnel in the world for any purpose. It is horseshoe-shaped and concrete-lined, with inside dimensions of 11 ft. 6 in. in height by 10 ft. 3 in. width, and has a uniform slope of 4.4 ft. per mile except for the northerly 3½ miles which is depressed, making that portion a pressure tunnel. Its capacity, computed as a grade tunnel, is 650 000 000 gal. daily.

The intake is located about 3½ miles north of Prattsville, N. Y. From this point the tunnel extends in a general southeasterly direction to just south of Allaben, N. Y., where it discharges into Esopus Creek. An intake shaft and seven intermediate shafts are provided, the aggregate depth of the shafts being 3 238 lin. ft., the maximum depth of a single shaft being 630 ft. The mini-

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in **September, 1927**, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

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mum distance between shafts is 1.3 miles, and the maximum, 2.7 miles. All shafts are circular with a diameter of 14 ft. inside the concrete lining. The upper portion of the intake shaft is so constructed that it will act as a Venturi meter, the building over this shaft containing the control gates and keeper's residence; discussion of the intake and outlet structures is not included in this paper.

Historical

Apparently, the first mention of a tunnel carrying water from Schoharie Creek to Esopus Creek was made in a paper published in the *Scientific American*, of September 4, 1886. Years later, in 1903, a survey was made for such a tunnel by the Commission on Additional Water Supply for New York City (William H. Burr, M. Am. Soc. C. E., the late Rudolph Hering, M. Am. Soc. C. E., and John R. Freeman, Past-President, Am. Soc. C. E.). The first engineering work on this project by the Board of Water Supply of New York City was done in 1907 when a line of levels was run. The surveys from which the final alignment of the tunnel was made, were run in 1916. The contract for construction was let on November 10, 1917, to the Degnon Contracting Company, and all work was completed on October 22, 1924. Thus, a period of 38 years elapsed from the time of the original published proposal to the final completion of the work.

Geology

In the region penetrated by the tunnel the rock was gray sandstone and red and gray shale (red predominating) lying in nearly horizontal layers, the dip being from 2½ to 2° in a southerly direction. The shales drilled easily, broke well to line, and would sometimes stand for a limited time, but they had a marked tendency to disintegrate when exposed to the atmosphere. Of slightly coarser texture, but often difficult to distinguish from the shales, were the red sandstones. Almost invariably the two were more or less mixed, and, when penetrated in the roof of the tunnel, needed support. The gray and blue sandstones, being the hardest and coarsest grained rocks in the vicinity, were more difficult to drill and shoot, but had an advantage in that they required little sealing and generally stood well without support when not too thinly bedded.

omit

During driving the temperature in different parts of the tunnel remained almost constant at 59° Fahr. despite the variation in cover from 200 to 2 200 ft. This constant temperature may have been due to the system of ventilation. Several crush zones were encountered between Shafts 3 and 4 (Fig. 1). At one of these a horizontal clay seam surmounted by a badly broken-up bedding of shales caused a maximum inflow of water of 170 gal. per min., which decreased in a few days to 65 gal. per min. A mud seam, varying in thickness from 5 to 7 ft., also crossed the tunnel line. Another crushed zone yielded an initial flow of 130 gal. per min., which rapidly dropped to less than 30 gal. This water was highly saline and smelled and tasted of sulfur. Apparently, it was of Devonic origin. In general the tunnel was remarkably dry, the largest quantity pumped at any one shaft being 170 gal. per min.

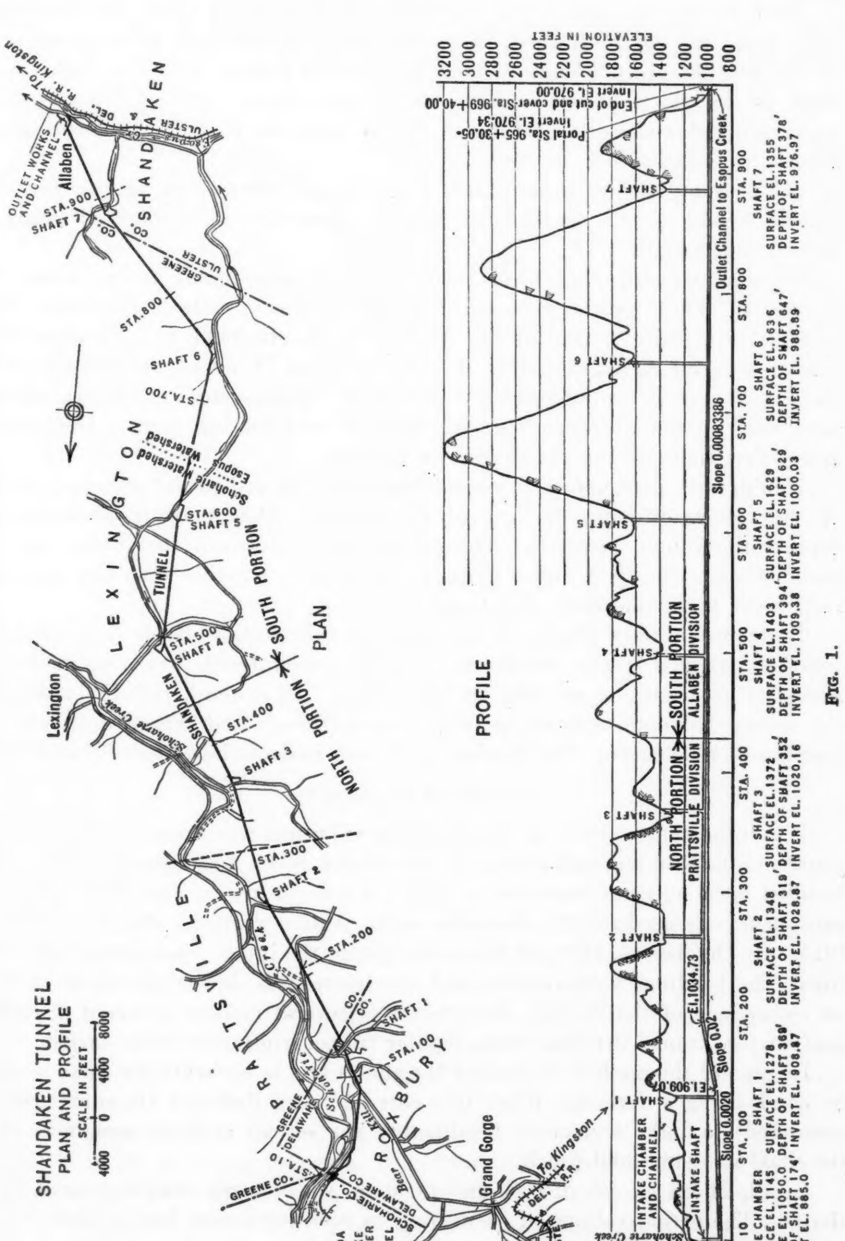

SHANDAKEN TUNNEL
PLAN AND PROFILE
SCALE IN FEET
4000 0 4000 8000
PLAN
PROFILE
NORTH PORTION
SOUTH PORTION
PRATTSVILLE DIVISION
ALLABEN DIVISION
ELEVATION IN FEET
Slope 0.00083386
Outlet Channel to Esppus Creek
Portal Sta. 965+30.05
Invert El. 970.34
End of cut and cover-Sta. 969+40.00
Invert El. 970.00
STA. 10
INTAKE CHAMBER
SURFACE EL. 1145
INTAKE EL. 1050.0
DEPTH OF SHAFT 174'
INVERT EL. 885.0
STA. 100
SHAFT 1
SURFACE EL. 1278
DEPTH OF SHAFT 369'
INVERT EL. 908.87
STA. 200
STA. 300
SHAFT 2
SURFACE EL. 1348
DEPTH OF SHAFT 319'
INVERT EL. 1028.87
STA. 400
SHAFT 3
SURFACE EL. 1372
DEPTH OF SHAFT 352'
INVERT EL. 1020.16
STA. 500
SHAFT 4
SURFACE EL. 1403
DEPTH OF SHAFT 394'
INVERT EL. 1009.38
STA. 600
SHAFT 5
SURFACE EL. 1629
DEPTH OF SHAFT 629'
INVERT EL. 1000.03
STA. 700
STA. 800
SHAFT 6
SURFACE EL. 1636
DEPTH OF SHAFT 647'
INVERT EL. 988.89
STA. 900
SHAFT 7
SURFACE EL. 1355
DEPTH OF SHAFT 378'
INVERT EL. 976.97

FIG. 1.

Between Shafts 4 and 5 the phenomenon of "popping rock" was observed. This rock was coarse-grained gray sandstone and appeared to be sound and of the best texture. "Popping" usually occurred shortly after the tunnel had been excavated; it ceased in from one to two weeks. A loud report would be heard, followed by the falling of a slab from the roof or the shooting of a rock fragment across the tunnel.

Small quantities of inflammable gas were encountered in the vicinity of Shafts 3 and 4 and occasionally minute quantities of carbonized organic matter were found.

Brachiopods and other forms of shell fossils were found in two strata 30 ft. apart. These strata were encountered on Stevens Mountain above the Gilboa Dam; back of the intake chamber, at the Bearkill Falls; and in the tunnel between Shafts 1 and 2. Fossil twigs and branches were also found, as well as a small stump similar to the fossil stumps found at Gilboa, which have been named *Eospermatopteris* (dawn of seed-bearing fern). Geologists assert that they are the oldest trees in history.

Preliminary sub-surface investigations disclosed pre-glacial gorges at each of the valleys crossing the line of the tunnel. This made it advisable to depress the tunnel below the hydraulic gradient between the intake and a point between Shafts 1 and 2 (Fig. 1) in order to provide sufficient natural rock cover under the Bear Kill Gorge.

From preliminary studies it was thought that comparatively little timber would be required; the conditions actually encountered, however, made it necessary to timber about 50% of the tunnel. In general, this included all the shales that disintegrated on being exposed to air and the areas in which popping rock occurred. The crushed zones required special types of timbering.

Construction Shafts

While the construction of the Shandaken Tunnel was done under a single contract, the sinking and lining of the shafts, being specialized work, were handled by a separate organization under a sub-contract. The shaft subway equipment was used for this operation only. Active work was started in June, 1918. By October 15, 1919, all the shafts except the intake were completed and lined; the headings were turned; and the tunnel was driven for about 35 ft. on either side of the shafts. An interim of several months occurred at each shaft before tunnel driving, using regular tunnel equipment, was started.

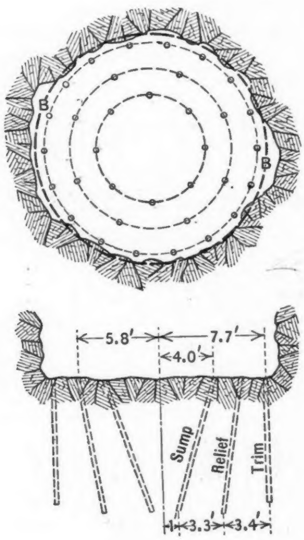
In general the method of sinking the shafts was to excavate the earth cover by hand, using a derrick. When this part had been timbered (in some cases, concreted), a light temporary head-frame was set up and the remainder of the shaft was sunk and lined.

In all, 24 521 cu. yd. of rock and 1 066 cu. yd. of earth were excavated for shafts. The actual volume of excavation in rock per linear foot of shaft was 7.65 cu. yd.; and the payment volume, 8.01 cu. yd.

The general method of drilling and shooting is shown in Fig. 2 and Tables 1 and 2. At some of the shafts the relief holes were omitted; this reduced the amount of drilling, also of powder, and gave equally good results.

There was a considerable variation at the different shafts in the number of holes drilled per round, in the depth of holes, and in the quantity of powder

used. These variations were frequently due more to the superintendent than to the character of the rock. In general, the number of holes varied from 30 to 38, the linear foot of hole per cubic yard of excavation from 5.3 to 6.5, and the powder from 1.3 to 3.2 lb. per cu. yd. The "pull" per round was between 5 and 8 ft.



DRILLS USED

MAKE	DIAM. OF STEEL	DIAM. OF BIT	LENGTH	REMARKS
Ingersoll	7/8"	1 3/4"	2' to 3'	Starters
Jack Hammer	7/8"	1 1/2"	5'	
Drills	7/8"	1 1/4"	8' to 10"	

Note:- 60% dynamite was used in sticks 8" x 1" diam., weighing ½ lb.
All holes drilled before any shooting or mucking done.
See Tables 1, 2 and 3 for added data.

SHANDAKEN TUNNEL
SHAFT EXCAVATION IN ROCK
METHOD OF DRILLING & SHOOTING

FIG. 2.

Supplementing the other data, Table 3 gives a more detailed analysis of excavation at Shafts 4 and 5. The variable in these two shafts was the percentage of shale. The shale drilled more easily and broke better, otherwise conditions at both shafts were about the same, and the difference in progress may be attributed to the difference in plant, that at Shaft 5 being much better than that at Shaft 4.

When any appreciable quantity of water was encountered in sinking the shafts, holes were drilled and grouted to shut off the flow. For example, while drilling a round of holes between Elevation 1254 and 1247 at Shaft 4, the flow of water, which had been no greater than 3 gal. per min., suddenly increased to 18 gal. per min. The sump was thoroughly cleaned and eleven equally spaced holes, as near the outer side of the excavation as possible, were drilled 9 ft. deep, pointing outward so that the lower ends were 3 ft. outside the ordinary limits of excavation. Pipes were wedged tightly into the holes and 68 cu. ft. of liquid grout, in the proportion of 9 gal. of water to 1 bag of cement, was forced in by means of a Canniff tank under a pressure of 70 lb. The flow practically stopped for the time being, but it broke out after mucking had reached Elevation 1236. Five horizontal holes were then drilled 3 ft. deep in a seam at Elevation 1244. Through these, 14 cu. ft. of grout was forced at 70 lb. pressure. This effectually stopped the leak.

After the shaft had been excavated to a depth of from 100 to 300 ft., depending on the character of the rock, it was lined with concrete starting from the bottom and working up. Simple, metal-covered, wooden forms were used, 5 ft. high in four 90° sections, with a key. Concrete was mixed at the top of the shaft, lowered in a bucket, and dumped on a platform from which it was shoveled into the forms.

TABLE 1.—PRATTSVILLE DIVISION—DRILLING AND SHOOTING DATA.

Description.	Order of shooting.	Number of holes.	Depth, in feet.	Linear feet of holes.	Powder Used, in Pounds. Hole.	Powder Used, in Pounds. Total.	Average Pull per Round. Linear feet.	Average Pull per Round. Cubic yards.	Powder used, in pounds, per cubic yard of excavation.
Holes, 7 Feet in Depth.									
Sump	1	8	7	56	2.33	18.7	..	..	..
Relief	2	8	7	56	2.33	18.7	..	..	..
Trim	3	18	6	108	1.82	32.7	..	..	..
Totals	..	34	..	220	..	70.1	5	39.6	1.77
Holes, 10 Feet in Depth.									
Sump	1	8	10	80	6.3	50.4	..	..	..
Relief	2	8	10	80	6.2	49.6	..	..	..
Trim	3	18	9	162	5.5	99.0	..	..	..
Totals	..	34	..	322	..	199.0	8.0*	63.4	3.15

FORCE EMPLOYED.

Shift.	Class of work.	Superintendent.	Shift Boss.	Mechanic.	Carpenter.	Hoist Runner.	Fireman.	Drill Runner.	Drill Runner Helper.	Blacksmith.	Blacksmith Helper.	Muckers.	Topmen.	Laborers.	Timekeeper.	Total.
12:00 M. to 8:00 A. M.	Drilling†	..	1	..	..	1	1	3	3	..	..	..	1	2	..	12
8:00 A. M. to 4:00 P. M.	Mucking	1	1	1	1	1	1	..	..	1	1	5	1	4	1	19
4:00 P. M. to 12:00 M.	Drilling†	1	1	..	..	1	1	3	3	..	..	..	1	2	..	13
Totals		2	3	1	1	3	3	6	6	1	1	5	3	8	1	44

* Not obtained at first shooting, it generally being necessary to blow out butts, reload, and fire a second time. This accounts for excessive use of powder.

† When drilling is finished before end of shift, drillers complete shift as muckers.

This mix was in the proportion 1:2.33:4.67, a bag of cement being estimated as 0.905 cu. ft., and no correction being made for swell in the aggregate due to moisture. The cement factor for concrete placed in the shafts averaged 1.57 bbl. per cu. yd. Fine aggregate came from local pits or crusher screenings; coarse aggregate was either crushed sandstone from the excavation or crushed field stone from neighboring stone walls. Grout was forced behind the lining at all water-bearing seams, by the method already explained.

The average progress for all shafts was 1.1 ft. per shift; and for lining, 6.1 ft. The average progress for sinking and lining was 8.3 ft. per week.

The plant at each shaft included a compressor with a capacity of from 275 to 528 cu. ft. of free air per min.; three to six jack-hammer drills; a hoisting engine; a crushing and screening plant; a boiler or boilers aggregating from 50 to 100 h.p.; a ½-yd. concrete mixer; a generator of from 2 to 3¾ kw.; and trucks and teams.

TABLE 2.—ALLABEN DIVISION—DRILLING AND SHOOTING DATA.
(Compiled by averaging data from Shafts 4, 5, 6, and 7.)

Description.	Order of shooting.	Number of holes.	Depth, in feet.	Linear feet of holes.	Powder Used, in Pounds.		Average Pull per Round.		Powder used, in pounds per cubic yard of excavation.
					Hole.	Total.	Linear feet.	Cubic yards.	
Sump	1	9	8½	76½	5—	43	....	....	....
Relief	2	9	8	72	5—	42	....	....	....
Trim	3	17	8	136	4+	70	....	....	....
Totals	....	35	....	284½	....	155	6.4	51±	3.04

FORCE EMPLOYED.

Shift.	Class of work.	Superintendent.	Shift Boss.	Mechanic.	Carpenter.	Hoist Runner.	Fireman.	Drill Runner.	Drill Runner Helper.	Blacksmith.	Blacksmith Helper.	Muckers.	Topmen.	Laborers.	Timekeeper.	Total.
12:00 M. to 8:00 A. M.	Drilling*	..	1	..	..	1	1	2	..	..	..	4	3	..	..	12
8:00 A. M. to 4:00 P. M.	Mucking	1	1	1	..	1	1	2	..	1	1	4	3	..	1	17
4:00 P. M. to 12:00 M.	Drilling*	1	1	..	..	1	1	2	..	..	..	4	3	..	..	13
Totals	....	2	3	1	..	3	3	6	..	1	1	12	9	..	1	42

* When drilling is finished before end of shift, drillers complete shift as muckers.

Small camps were built at each shaft and enlarged before tunnel driving started; these will be described later. Quote later

TUNNEL CONSTRUCTION

The tunnel was driven both ways from all shafts except Shaft 2 (Fig. 1). A short length of tunnel was driven from the outlet, in earth, but none from the intake.

In general three methods of driving were used: The top heading, the full-face heading, and the bottom heading. In sound rock most of the tunnel was driven with a top heading with a bench from 40 to 50 ft. long. In the earlier stages of the work the bottom heading was used, but this was abandoned later, when the work encountered unsound rock, which seemed better adapted to the full-face heading. In unsound rock, where support was required up to the face, the full-face heading, combined with a mechanical mucker, permitted the most satisfactory progress.

TABLE 3.—Details of Work at Shafts 4 and 5.

Item.	Shaft 4.	Shaft 5.
Character of rock:		
Percentage of shale	25	52
Percentage of sandstone	75	48
Progress:		
Date started	October 21	January 2
Date finished	February 4	February 18
Time consumed, in days	107	48
Dimensions:		
Elevation, top, in feet	1 389	1 418
Elevation, bottom, in feet	1 140	1 236
Depth, in feet	249	182
Quantities:		
Payment, in cubic yards	±2 023	±1 411
Actual in cubic yards	±1 915	±1 283
Percentage, excess of actual	6	12
Excavation, per day:		
In linear feet	2.3	3.8
In cubic yards	18	27
Sump:		
Number of holes	± 10	± 11
Depth, in feet	± 8.5	9.5
Radius, in feet	5	6
Charge, in sticks of dynamite	85	113
Relief:		
Number of holes	± 10	*
Depth, in feet	± 8.5	....
Radius, in feet	6.33	....
Charge, in sticks of dynamite	85	....
Trimming:		
Number of holes	± 18	± 18
Depth, in feet	± 8	8
Radius, in feet	7.58	7.67
Charge, in sticks of dynamite	144	133
Round:		
Number of holes	± 38	30
Depth of excavation, in feet	6 to 7	7
Charge, in sticks of dynamite	314	246

* Only one set of relief holes was drilled. This round consisted of 8 holes, each 8 ft. deep; 80 sticks of dynamite were used.

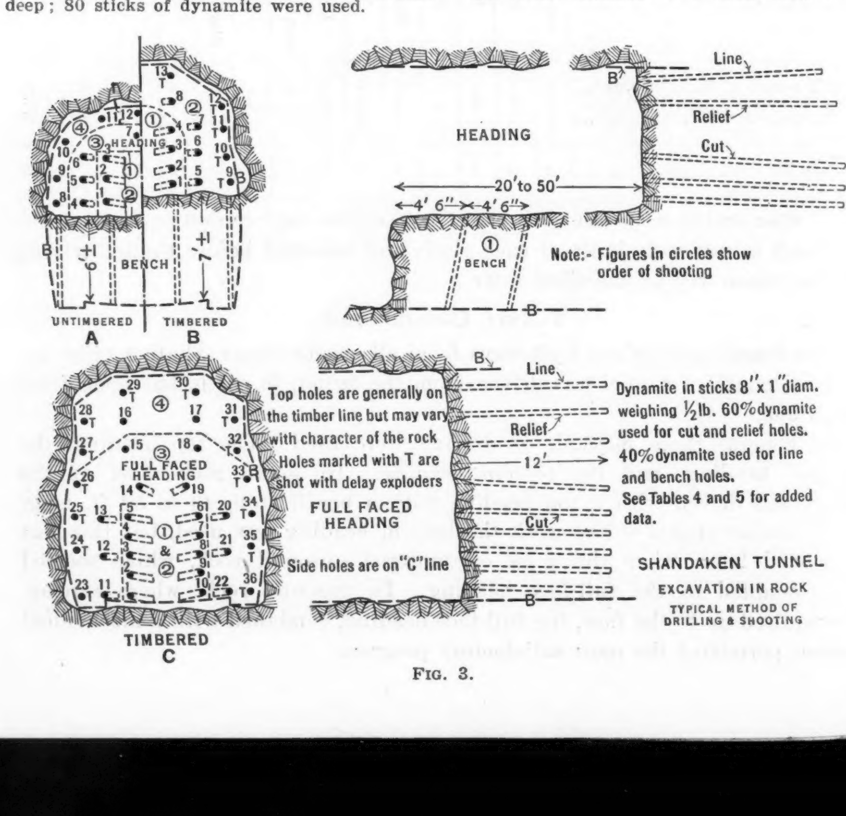


Fig. 3.

Details of the average methods used in these two types of driving are shown on Fig. 3 and in Tables 4 and 5. Vertical columns for supporting the drills were used in both methods.

Drills.—In the headings Ingersoll-Rand water Leyner drills No. 248 were used, and on the bench, Jap drills, B. C. R. No. 430. After numerous tests the six-point, or rose, bit was adopted. This bit has a flat taper for a short distance back from the face, which prevents the points from breaking. After each round the drill steel was sharpened with a special machine. The drills were carried back from the heading, taken apart, cleaned and greased, and made ready for the next round. As it was found difficult to make the drillers keep their machines oiled, a 1-gal. oil cup was placed on the air-line and regulated in such a manner that a predetermined quantity of oil would be fed to the drills with the air. An expert drill man spent his entire time going from shaft to shaft, inspecting and repairing the drills so that they might be kept at maximum efficiency.

Scaling was done after each shot and at regular intervals throughout the length of the untimbered tunnel. Except for that done immediately after blasting, little was necessary.

Trimming.—Once a week the trimming was determined and marked. In general the tunnel was kept trimmed at all times well toward the heading and down to 1 ft. above the invert grade.

Muck.—In the top-heading method, muck was shoveled into wheel-barrows, wheeled along a runway supported on pipes extending across the tunnel, and dumped into muck cars—Sanford end dumpers—holding about 1 cu. yd. of solid rock. Trains of six or seven cars were hauled to the shaft; cars were taken up singly to the tipple and dumped into Western 4-yd. dump cars, which were pushed to the dump by electric locomotives. Each shaft was equipped with two cages so that when a full car was going up an empty one was going down.

The tunnel locomotives were of the storage battery type, the greater number being General Electric, 4-ton capacity, 30-in. gauge, with a drawbar pull of 1 000 lb., a speed of 5 to 6 miles per hour, and using a voltage of 85 to 125. The batteries were readily detachable, extra ones being kept in the tunnel, where they were charged.

For the full-face heading a Myers-Whaley mucking machine was used to load the cars. This lifted the muck on to a traveling belt which emptied into the muck car. As each car was loaded it was placed on a near-by siding until the whole train was loaded.

Support.—It was a difficult matter to decide, in advance of excavation, the type of timbering that would best suit the conditions. In general, the ground did not develop heavy pressure, as evidenced by the rather wide ($7\frac{1}{2}$-ft.) spacing of arch ribs; yet when allowed to stand for a considerable period, the rock frequently disintegrated and "reeded". The use of temporary timber at such a place would have necessitated considerable additional excavation preparatory to placing concrete. This work is slow and expensive; and interferes greatly with lining operations. It was the rule to place permanent tim-

TABLE 4.—DRILLING AND SHOOTING DATA, ROCK TUNNEL (SEE FIG. 3).

Description.	Location.			Number of Holes.			Depth of Holes, in Feet.			Linear Feet of Holes.			Number of columns.	Number of Drills.			Average Time to Drill, in Hours.			Feet Drilled per Hour.			Pounds of Powder Used. Hole.			Pounds of Powder Used. Total.			Average Pull per Round. Linear feet.			Average Pull per Round. Cubic yards.			Pounds of powder used per cubic yard of excavation.		
	A.	B.	C.	A.	B.	C.	A.	B.	C.	A.	B.	C.		A.	B.	C.	A.	B.	C.	A.	B.	C.	A.	B.	C.	A.	B.	C.	A.	B.	C.	A.	B.	C.	A.	B.	C.
Heading. Cut...	1 to 3	1 to 4	5 to 10	6	8	10	13	12	12	78	96	120	2	....	....	....	....	....	....	....	....	....	14.9	11.4	8.1	80	91	81	....	....	....	....	....	....	....	....	...
Heading. Relief.	4 to 7	5 to 8	11 to 22	7	8	12	12	11	12	84	88	144	2	....	....	....	....	....	....	....	....	....	6.3	7.6	6.3	44	61	88	....	....	....	....	....	....	....	....	...
Heading. Line..	8 to 12	9 to 13	23 to 35	9	10	14	12	10	12	108	100	168	2	....	....	....	....	....	....	....	....	....	7.2	7.7	6.0	65	77	84	....	....	....	....	....	....	....	....	...
Total....	...	...	...	22	26	36	....	....	....	270	284	432	2	4	4	6	3.0	8.1	3.1	22.5	22.9	23.2	....	....	....	198	229	258	10.2	8.7	9.8	33.3	32.5	65.3	5.9	7.0	3.9
Bench. Untimbered, A....				8			7½			60			....	2			4			7.5			5.9			47.2			8.6			22.7			2.1		
Bench. Timbered, B......				8			9			72			....	2			4.2			8.6			6.1			48.8			8.5			24.6			2.0		

TABLE 5.—TYPICAL FORCE EMPLOYED FOR ONE SHIFT, ROCK TUNNEL.

Class of work.	Superintendent.	Walker.	Timekeeper.	Janitor.	Electrician.	Mechanic.	Pipeman.	Hoist Runner.	Compressor Engineer.	Motorman.	Brakeman.	Signal Man.	Dumpman.	Blacksmith.	Blacksmith, Helper.	Muck Boss.	Muckers.	Muck Machine Operator.	Heading Boss.	Drill Runner.	Drill Runner, Helper.	Powder Man.	Nipper.	Track Boss.	Track Man.	Timber Boss.	Timber Man.	Timber Helper.	Top Man.	Total.	Location.
General.........	1	1	1	1	1	1	2	1	1	1	..	2	1	1	1	..	..	..	..	..	..	..	..	..	..	..	..	..	2	18	Two headings
Mucking.........	..	..	..	..	..	..	..	..	..	2	1	2	..	..	..	1	25	*	..	..	..	..	..	..	..	..	..	..	..	31	One heading
Drilling.........	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	1	6	6	1	1	..	..	..	..	..	..	15	One heading
Timbering......	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	1	3	2	..	6	One heading
Track gang.....	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	1	6	..	..	..	..	7	Two headings

* Where mucking machine is in operation only two muckers and one mucker machine operator are employed.

ber wherever the roof was in soft red shale, or in thinly bedded sandstones, or where "popping rock" occurred. Where support was needed merely to protect men from the small thin slabs that fell, temporary timber was placed. About 570 cu. yd. of rock, or an average of 0.1 cu. yd. per lin. ft., was removed with the temporary timber. This was not evenly distributed; where the temporary support was replaced with a permanent one the amount was greatly in excess of this average.

The usual type of support was a three-piece arch rib of 10 by 10-in. timbers, spaced $7\frac{1}{2}$ ft., center to center, with 3 or 4-in. lagging. It was supported on each side on a short 3-in. plate resting on three $1\frac{1}{4}$-in. steel pins, 24 in. long, set 18 in. in the rock. From four to seven 2 by 10-in. planks (known as "lacing") were spiked to the under side of the arch ribs to provide lateral support. Extra lacing was placed near the heading to protect the timber.

The same type of construction was used for both temporary and permanent timber (Fig. 4), the permanent timber being placed above the normal concrete lining while the temporary timber was lower. The space between the permanent timber and the roof of the tunnel was packed with rock. Ordinarily, temporary timber was left without packing, but in case of high breakage cordwood was used.

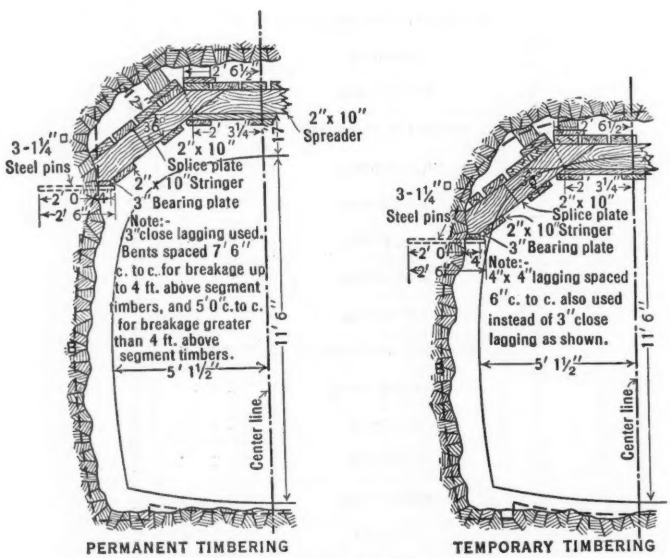


FIG. 4.—STANDARD TIMBER ROOF SUPPORT, SHANDAKEN TUNNEL.

Lumber for crown pieces, lagging, and lacing was cut to dimensions in the Allaben Yard. A special device made it possible to measure the length of legs accurately in the tunnel and then cut them on top, thus saving labor and transportation. The timber used was miscellaneous hard and soft woods. All varieties showed a marked tendency to decay; and much of it that had been in place for two or three years was punky and lifeless. Due to this, some bents fell and others had to have supplementary supports for the lagging.

At other places the lagging failed. In all, thirty-one bents were replaced or supplemented. While this is a very minute percentage of the total number of bents, it certainly shows that it is not safe to rely on timber support for too long a time.

Progress of Timbering.—Approximately 3 083 000 ft. B. M. of lumber was placed in the permanent support and 363 000 ft. B. M. in the temporary support. About 6 700 lin. ft., or 7% of the length of the tunnel, was driven with the temporary support. Approximately 1 000 lin. ft. of this was later replaced with permanent timber; this, with the 38 950 lin. ft. driven as a permanently supported type, gave a total length of 45 650 lin. ft. of timbered tunnel, or 47.7% of the total length.

The force on the timbering included 1 carpenter and 2 laborers cutting, on top of the shaft; and 1 foreman, 3 carpenters, and 4 laborers erecting in the tunnel. This gang could make an average progress of 20 lin. ft. completely erected and dry-packed in 8 hours.

Explosives.—Various well-known brands of explosives were tried. Finally, Dupont gelatine low-freezing dynamite was adopted, as this appeared to give the smallest amount of objectionable fumes. A special wrapper, developed by the powder manufacturer for use on this job, was very effective in reducing objectionable fumes. A saving was effected by using 60% dynamite for cut holes and 40% for all others. A slight saving was also effected by having the size of the sticks slightly reduced, as the loader measured his charge by the number of sticks and not by weight. In general, cut holes had to be fired more than once and occasionally three times on account of the depth.

Dupont electric blasting caps (No. 8) fired from a switch on the electric circuit, were used for exploders. First delay exploders were used for line holes. Some second delay exploders were tried, but for various reasons were finally discarded. A small amount of shooting was done with a fuse, using No. 6 fuse caps for exploders. This burned at the rate of 1 ft. per min. Two men lit the fuse, one on each side of the face. Two reasons for not using fuse more generally were the lack of experienced men and the improvement of delay exploders.

Tamping material consisted of crusher screenings or natural sand put up in paper bags 12 in. long and $\frac{1}{14}$ in. in diameter. Brass or copper pipe was used to blow out holes that had once been loaded.

Storage for 400 to 10 000 lb. of explosives was provided at each shaft in specially constructed magazines, built to conform to the rules of the State Industrial Code. There was also a small magazine near the shaft head for storage between loading periods.

Power.—Electric power was purchased from outside sources. This involved the construction of 48 miles of line. Wooden poles, 35 to 50 ft. long, spaced 150 ft. apart, having two wooden cross-arms with three bare 7-strand, No. 1 copper wires, were used to carry the current to the nearest shaft. A similar construction but with No. 4 wire was used along the tunnel line. Every fifth pole was grounded with 7-strand, galvanized, messenger wire. The cur-

rent was 60-cycle, 3-phase, 33 000 volts. At each working point it was stepped down to 440, 220, and 110 volts.

The power consumption during 1922, when both driving and lining was in progress, was about 11 700 000 kw-hr., including line loss. During January, February, and March of that year, when driving only was in progress in all twelve headings, the power consumption was 2 758 000 kw-hr., or 2 554 kw-hr. per heading per day. During this period 95 000 cu. yd. of rock were excavated at about 30 kw-hr. per cu. yd., not including line loss. The mucking machine required about 1 kw-hr. to load a car, taking about 3 min.

A 71-hour power test made during this period showed an average power consumption of 1 328 kw. per hour, with a maximum of 1 840 and a minimum of 960 kw. Drilling and shooting were in progress in eleven headings about 5% of the time, in six headings about 50% of the time, and in one heading about 90% of the time. This test showed that from 22.2 to 36.8 kw-hr. was required for each cubic yard of rock taken from the tunnel, with an average of 27.3 kw-hr.

Ventilation.—The plenum process was used. A Connersville blower with a capacity of 2.77 cu. ft. of free air per rev. was driven by an electric motor at speeds sufficient to supply from 900 to 1 200 cu. ft. of compressed air at from 7 to 10 lb. pressure. The air was conveyed down the shaft by an 8-in. pipe and through the tunnel in a 6-in. steel well-casing about ⅛ in. thick, weighing 11.65 lb. per ft. and supported on rods driven into the sides of the tunnel about 5 ft. above the invert.

The pipe was made with Dresser joints. This joint is flexible, requires no threading, and permits the use of comparatively thin pipe; it is quickly and easily repaired or replaced. During the shooting period, and for a short time afterward, the 4-in. air-line used for drilling, was opened at the heading. This helped to clear the heading rapidly and drive the smoke cloud toward the shaft. Generally speaking, the ventilation was satisfactory except that smoke clouds occasionally interfered with alignment work in the longer headings.

Compressed Air.—Two compressors, each rated at 750 cu. ft. of free air per min., and electric-driven, furnished air for the drills. A 4-in. steel well-casing, weighing 6.06 lb. per ft., and suspended on rods driven in the side of the tunnel, carried the air to the heading. This pipe was also made with Dresser joints. Air was furnished to the drills at about 90 lb., varying with the length of the tunnel. A manifold at the end of the line served the drills. There was always sufficient compressed air.

Pumpage.—Water was either drained by gravity or was pumped by air or electricity to a sump at the foot of the shaft, whence it was pumped to the surface. A typical installation for this was a Gould triplex, 5 by 8-in. pump, rated at 120 gal. per min., driven by an electric motor delivering water to the sump; and a Cameron pump with a 5-in. water cylinder, 10-in. air cylinder by 13-in. stroke, rated at 125 gal. per min., raising it to the surface. At the top of the shaft the water was measured over a V-notch weir equipped with an automatic recording gauge. The inflow of water was never great enough

to influence progress, the maximum in any heading being 170 gal. and the minimum, 7 gal. per min. The total pay pumpage was 162 545 000 000 ft.-gal.

Illumination.—A 220-volt current was used for lighting the shaft and tunnel, the spoil area, and parts of the camp. The current was carried into the tunnel by three No. 0000-gauge cables supported on pins driven in the side of the tunnel about 8 ft. above the track. Attached to these cables from 40 to 50 ft. apart were 50-watt lamps. Within a few hundred feet of the heading the voltage was reduced to 110 by a small portable transformer; a portable cable, No. 12 gauge, carried current for lights in the heading.

Progress.—In general, some work was in progress throughout the 24 hours. Two methods were used. In one, the drilling and mucking shifts overlapped four hours and two advances were made in a heading each day; in the other, known as the swing shift, two advances were made in the north heading and one in the south on one day, and one advance was made in the north heading and two in the south on the next day. A law preventing men from working more than 8 hours per day was in force until May 25, 1923.

In all, 613 668 cu. yd. of rock were excavated in the tunnel. The quantities of excavation, in cubic yards per linear foot, for the two principal types of tunnel, were, as given in Table 6.

TABLE 6.

Quantities.	Unsupported.	Supported.
Theoretical	5.71	6.61
Actual	6.09	6.91

The unsettled condition affecting the country generally, as a result of the World War, had its effect on the progress of the work and the cost to the contractor. On November 11, 1920, the Degnon Contracting Company made an assignment to the Shandaken Tunnel Corporation for which the active agent was the Ulen Contracting Corporation. At the time the contract was assigned the work was approximately a year behind schedule. The job, however, had been well equipped by the Degnon Contracting Company. With new capital and efficient management the new organization succeeded in overcoming the time handicap and completed the underground work four months ahead of schedule. This is shown effectively on Fig. 5, which indicates the rates of progress for the principal items. The excavation by years is given in Table 7.

The average monthly progress of excavation per heading, including the time when, for various reasons, work was not in progress, was 280 ft. In headings, where work was more nearly continuous, the average progress with two drilling and two mucking shifts was about 417 ft. per month, with a maximum of 612 ft. The average "pull" per round in the various headings varied from 7.5 to 10 ft.

Bonus.—During 1921 the contractor worked out a bonus system which was successful in reducing costs and increasing progress. A bonus was given

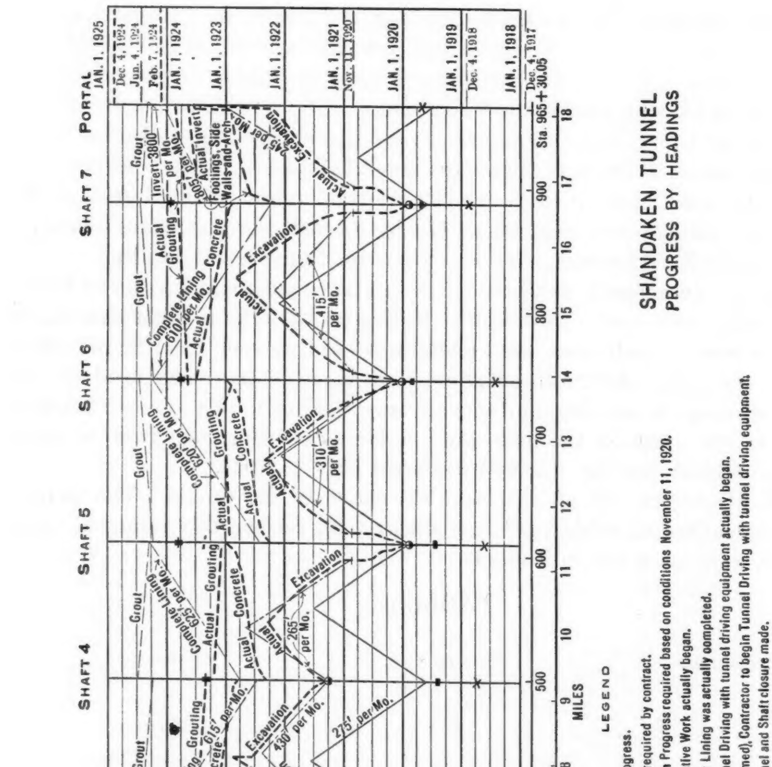

SHANDAKEN TUNNEL
PROGRESS BY HEADINGS
INTAKE
SHAFT 1
SHAFT 2
SHAFT 3
SHAFT 4
SHAFT 5
SHAFT 6
SHAFT 7
PORTAL
JAN. 1, 1925
Dec. 4, 1924
Jun. 4, 1924
Feb. 7, 1924
JAN. 1, 1924
JAN. 1, 1923
JAN. 1, 1922
JAN. 1, 1921
Nov. 11, 1920
JAN. 1, 1920
JAN. 1, 1919
Dec. 4, 1918
JAN. 1, 1918
Dec. 4, 1917
Sta. 10
Sta. 965+30.05
MILES
LEGEND
Actual progress.
Progress required by contract.
Excavation Progress required based on conditions November 11, 1920.
Date Effective Work actually began.
Date Shaft Lining was actually completed.
Date Tunnel Driving with tunnel driving equipment actually began.
Date (assumed), Contractor to begin Tunnel Driving with tunnel driving equipment.
Date Tunnel and Shaft closure made.
NOTES-
Dec. 4, 1924 - Contract to be completed.
Oct. 22, 1924 - Contract completed.
Jun. 4, 1924 - Sub-surface work to be completed.
Feb. 7, 1924 - Sub-surface work completed.
Nov. 11, 1920 - Contract assumed by the Shandaken Tunnel Corporation.
Dec. 4, 1918 - Effective work to be started at all points.
Dec. 4, 1917 - Notice to begin work served.

FIG. 5.

to the superintendent and walking bosses at each shaft, provided the costs per cubic yard of excavation were decreased below a fixed figure. Likewise for each shaft, the drilling foreman whose gang made the highest average "pull" per shot for the week received a bonus, while a similar bonus was distributed to the gang in proportion to their pay.

TABLE 7.—PROGRESS IN EXCAVATING SHANDAKEN TUNNEL.

Year.	Progress in linear feet.	Remarks.
1919........	1 095	Shaft sinking completed and tunnel excavation started October 27 at one point.
1920........	12 041	Contract assigned November 11.
1921........	47 363	Steady acceleration of progress.
1922........	33 435	All but one heading "holed through" during year.
1923........	1 134	Finished excavation February 13.

The effect of this bonus on progress was very marked. Before installing the system the average "pull" was 7.5 ft. Three weeks after the bonus had gone into effect the average "pull" had increased to 8.18 ft. and within five months it had reached 9.34 lin. ft.

Earth Tunnel.—At the south end the roof of the tunnel for 534 lin. ft. was in earth. At the junction with the rock the earth was a compact mass of clay, sand, and boulders, with little or no water. No difficulty was experienced in driving this part of the tunnel.

The support for the earth part of the tunnel driven from Shaft 7 was a 5-bar arch, 5 ft. on centers, supported on wall-plates. The methods of excavation were similar to those used in rock including, to some extent, the use of light charges of explosives. The advance was made with a top heading kept from 10 to 70 ft. ahead of the bench. About 240 ft. was successfully driven in this manner. However, as the heading advanced pockets of blue clay were encountered and, later, the material changed to a saturated mixture of clay, sand, and gravel of the consistency of wet concrete. Driving became difficult and the full heading was abandoned; wall-plate drifts, or a center drift, or a combination of both, were substituted. The spacing of bents was reduced to 3 ft. and then to 2 and 1½ ft. Breasting was necessary for the full drift section. After a fair trial this method was abandoned on account of small progress.

Following the cessation of driving from Shaft 7 an attempt was made to drive the tunnel from the portal. A small shaft was sunk and a drift started, but this soon failed due to insufficient breasting. A new shaft, 16 by 16 ft., was then sunk and a 9 by 8-ft. center drift, with its base at the wall-plate, was started. The cap of this drift was in line with the crown of the completed three-piece rib section driven from the portal. This drift was advanced continuously with a consistent progress of about 25 ft. per week until it joined with the tunnel driven from Shaft 7.

Operations were carried on systematically, as the ground was very treacherous. The face was always breasted. In some places the ground was so

bad that only 6 or 8 in. could be opened at a time; however, no ground was lost and only a small portion of the timber settled, the maximum being 4½ in. All open spaces outside the lagging were carefully packed with salt hay or excelsior. Where clay was encountered in the roof the poling boards were spaced from ½ in. to 2 in. apart, so that the clay might swell without breaking the boards or caps.

After this drift had connected with the completed tunnel it was widened out to the full section down to the wall-plate, working south, and, later, the full excavation was completed, working in the same direction. The diagrams, Figs. 6, 7, and 8, give some of the details of this work.

The force working from the portal was on a three-shift basis. It included a superintendent and, on each shift, 2 to 3 miners, 2 to 3 helpers, 1 hoist engineer, and 1 pump man. Four shifts were required for one complete 5-ft. advance of the top drift. Of the top drift 220 ft. was completed in 65 days, while 52 ft. of re-drifting (in making the connection through the last advances made from the north) took 21 days. Enlarging the top drift to full size took 68 days. Removing 299 ft. of bench and completing the tunnel to full size took 114 days. The advance of all work, including drifting, widening, and taking out bench, was approximately 1 ft. per day.

In all, 3669 cu. yd. of earth were removed from the tunnel, an average section containing 8.57 cu. yd. per lin. ft.

Concrete.—The tunnel was lined with concrete throughout, placed from Shafts 1, 3, 4, 5, and 7. It was mixed on the surface, poured through a pipe into cars at the foot of the shaft, and hauled to the forms. The invert and side-walls were placed by hand, while the arch was placed with a "concrete gun". The general methods and sequence of operations are shown on Fig. 9.

The work progressed away from the shaft. Operations 1 and 2, and 3 and 4, (Fig. 9) were later combined by suspending the forms for the footing course and concreting the half invert and footing in one operation. On part of the work, 150 ft. of side-wall and arch forms were used. Templates for the half invert were set about 15 ft. apart. After the concrete had been screeded and rolled these templates were removed, and the empty space was filled with concrete. Pipes were set in the invert about 50 ft. apart to provide a vent for the relief of any pressures that might accumulate.

The fine aggregate consisted entirely of particles of crushed rock, as there was no acceptable natural sand in the vicinity. Part of this material came from the crusher, but most of it was made with sand rolls. The material proved satisfactory except that it caused difficulty in securing a smooth invert. The average fineness modulus was about 3.2. The amount passing the 100 sieve varied from 5 to 11 per cent. The coarse aggregate came from sandstone taken from the tunnel and from quarries opened up near the shafts. Shale was not used.

The concrete mix was $1 : 2\frac{1}{3} : 4\frac{2}{3}$ by volume, 1 bag of cement being assumed as 0.905 cu. ft. Measurement was made in pyramid-shaped hoppers, all materials being measured loose without correction for moisture. During

FIG. 6.—EARTH TUNNEL, 9 BY 8-FOOT CENTER DRIFT, THREE-BAR TYPE.

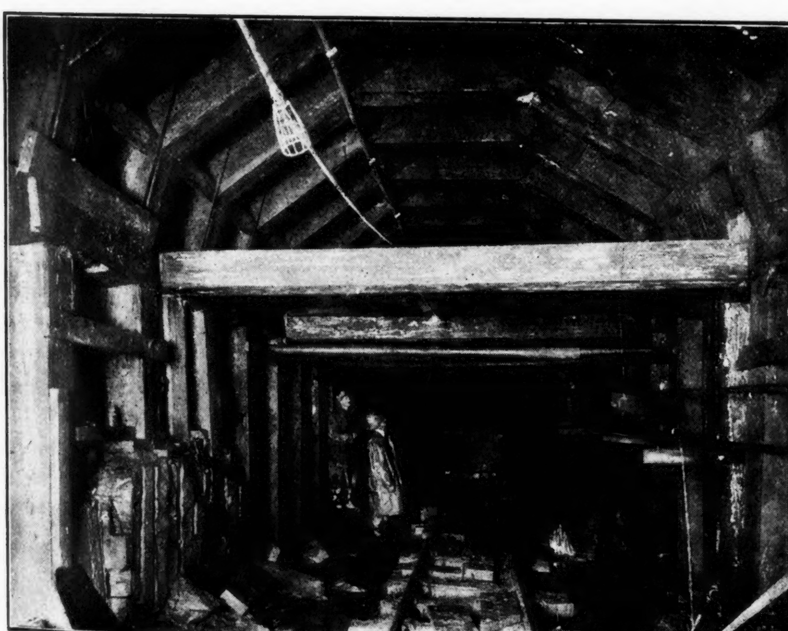


FIG. 7.—EARTH TUNNEL, COMPLETED TIMBER LINING, FIVE-BAR TYPE.

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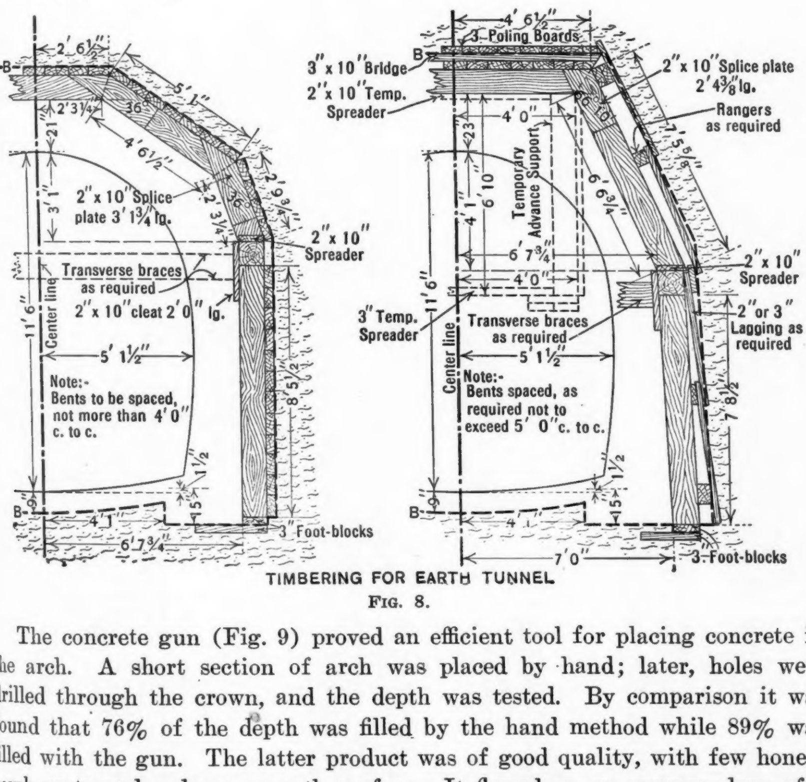
cold weather the sand and stone were thawed out by placing steam pipes in the bins, approximately 100 sq. ft. of radiation being supplied to each bin. The water was heated with live steam. The cement factor was 1.48 bbl. per cu. yd. of concrete. Empty cement bags were shaken by machine, with a recovery of 1 bag of cement to every 170 bags shaken. In general, sufficient water was added to make a plastic mix. The average compressive strength determined from twenty-eight representative cylinders was 1 484 lb. per sq. in. at the age of 28 days.



TIMBERING FOR EARTH TUNNEL

FIG. 8.

The concrete gun (Fig. 9) proved an efficient tool for placing concrete in the arch. A short section of arch was placed by hand; later, holes were drilled through the crown, and the depth was tested. By comparison it was found that 76% of the depth was filled by the hand method while 89% was filled with the gun. The latter product was of good quality, with few honeycomb spots and a dense, smooth surface. It flowed on an average slope of 1 on 13. When working at a distance from the shaft the gun required about 1 200 to 1 500 cu. ft. of air to operate it. It was possible to shoot a 6-car load, or 5½ cu. yd. of concrete, in 10 min.

The force placing concrete included:

7 men transporting materials on the surface;
1 foreman and 7 men operating mixer;
4 motormen and 1 signalman in tunnel transporting concrete;
1 foreman and 10 men placing concrete invert or footings*;
1 foreman and 11 men placing concrete side-wall*;
1 foreman and 9 men placing concrete arch*; and
1 foreman and 4 to 6 laborers on forms.

* Only one of these three operations was carried on at a time.

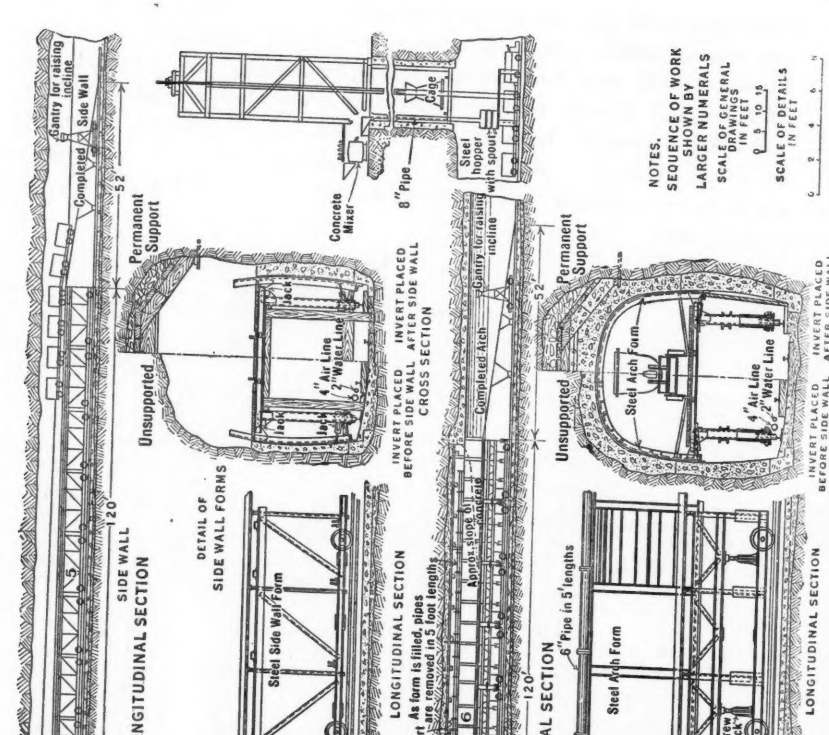


FIG. 9.—SECTIONS SHOWING CONCRETING OPERATIONS, SHANDAKEN TUNNEL.

Using two side-wall forms and two arch forms in the tunnel between Shafts 5 and 6, 120 ft. of lining was completed in 24 hours with three shifts. The average progress from Shaft 4 to Shaft 5 was 58.7 lin. ft. per day; and from Shaft 5 to Shaft 6, 59.4 lin. ft. This is a fair average for the entire job. In general, the rate of progress was limited by transportation.

In the lining, 212 677 cu. yd. of concrete were placed. The quantity per linear foot varied from 2.109 to 3.378 cu. yd. The quantity of pay concrete was only 183 564 cu. yd., because the contractor preferred driving a slightly enlarged tunnel to trimming. He was paid a nominal sum for the excess concrete plus the cost of the extra cement.

Grouting.—Liquid grout under low pressure was placed between the lining and the rock throughout the supported tunnel, and in all but 21 942 lin. ft. of the unsupported tunnel. The purpose of the grout was to bind the lining solidly to the rock, all voids being filled. The lining thus forms an air-tight cover around the permanent timber to arrest decay.

During the placing of the permanent timber, 2-in. pipes were set, extending through the lagging approximately to the roof of the tunnel. These pipes were near the center line, approximately 50 ft. apart. Before placing the concrete, 1½-in. pipes, extending down to the forms, were telescoped inside these pipes, and securely wedged. In addition, grout pipes were placed aproximately 50 ft. apart on either side of the tunnel extended through the lagging near the lower limits of the dry packing. The lower end of each pipe was fitted with a coupling that was kept about ½ in. from the forms, by means of a screw-plug, or what was more successful, by means of a bit of oily waste, which aided in locating the hole after the removal of the forms.

In the untimbered part of the tunnel pipes were placed along the axis at intervals of about 40 ft. and also at points of especially high breakage. These pipes were held in place by being screwed to a special flange attached to the key-plate. In special types of timber with tight lagging behind the plumb-posts, grout pipes were placed through the lagging.

Where water was encountered the leaks were caulked, or, if they extended over a considerable area, they were covered with tin. In either case the water was collected and led into the tunnel through a pipe usually placed on top of the side-wall. No attempt was made to shut off this leakage as the water was of good quality; but, in general, most of the leakage was stopped by grouting.

Grout was placed either with Canniff machines or with the concrete gun. After a sufficient trial it was found that the gun was not sufficiently mobile, and it was abandoned. In the early operations materials were mixed inside the tunnel with the Canniff machine. As grout could not be made in the gun it was mixed on the surface and transported to the gun. Although the gun was abandoned, the practice of mixing on the surface was continued, either 5 or 10-bag batches being made in the Ransome mixer in the proportions of 1 bag of cement, 1 bag of sand (90 lb. net), and from 5.5 to 7 gal. of water, with an average liquid volume of 1.92 cu. ft. per bag.

The mixture was poured down an 8-in. pipe into concrete cars at the foot of the shaft. The cars were hauled in trains of from three to six

cars to the Canniff machines. The grout was then pumped to a small tank from which it flowed by gravity into the Canniff tanks. During the pumping, the material in the cars was agitated continuously. The tanks forced the grout out at a pressure of from 30 to 75 lb. in rock; and at less than 20 lb. in earth. The grout traveled a maximum distance of 700 ft. in the timbered tunnel with an average of 200 ft. In the untimbered tunnel, the maximum travel was 470 ft., and the average, 50 ft. Connection was usually made to a low hole, and when a good flow of grout appeared in an adjacent high hole, that hole was plugged.

A natural fine sand, obtained from local pits, was used for grouting. When screened to remove the coarse particles this sand had a fineness modulus of about 1.6. Only about 18% was coarser than the No. 30 sieve, and 60% coarser than the No. 50. It was all finer than the No. 8 sieve.

Contraction cracks were found in the tunnel from 15 to 30 ft. apart, extending across the arch and down the side-wall. They were very noticeable when grouting was in progress, as were the horizontal and vertical construction joints. It is thought that all these cracks have been effectually closed by grouting.

The force for a single shift was 12 men on the surface and from 9 to 12 men inside the tunnel. At some shafts the outside force mixed grout for two headings. Grouting was done on a 1, 2, and 3-shift basis. A single shift placed on an average thirty-five 10-bag batches per 12 hours in unsupported tunnel. In supported tunnel a single shift averaged about ninety 10-bag batches in 10 hours. In the Allaben Division, where most of the grout was placed, the maximum linear feet of tunnel grouted per week was 4 000 in unsupported tunnel and 2 400 in supported tunnel; the average was 1 500 and 1 000. An average of 13.4 cu. ft. of liquid grout per linear foot of tunnel was placed in the supported type and 2.8 cu. ft. in the unsupported. The maximum quantity of liquid grout placed from one connection was 9 650 cu. ft. A total of 23 449 cu. yd. of grout was placed, at 3.5 bbl. of cement per cu. yd.

Cleaning

After grouting was completed the tunnel was cleaned. The first operation, working against the flow, was to remove the lumber, forms, clay, and muck that had accumulated on the invert. This allowed a fairly free passage for the flow of water. The side-walls were then scraped, using long-handled chisels with a 3-in. blade. Lips formed by the junction of forms were removed and the very few places that were honeycombed were cut out and patched.

The arch was then scraped from a high platform placed on a flat car, with the same tool used for the side-walls. Plugs were removed from grout pipes and the ends of the pipes were packed with mortar. Finally, the rails and ties were removed, and the invert was given a final scraping and brooming, working with the flow. The pipes left at 50-ft. intervals in the invert were cleaned out, or, when these could not be found, holes were drilled through the invert.

In the shafts all cage timbers, pipes, etc., were removed, all bolt holes plugged with mortar, and the sides scraped, after which the head-frame was removed. A concrete cover with a metal grating was then placed over the shaft.

Two shifts of about 10 men each, working 10 hours per day, made an average progress of 125 lin. ft. of finished cleaning in the tunnel (including all operations) per day.

Intake and Outlet

The intake is an open channel through rock to a down-take shaft equipped with gates for controlling the flow, and a vertical Venturi meter. The outlet is through short lengths of standard and pressure aqueduct to a water-seal, thence through an open channel to Esopus Creek. Both are specialized structures, beyond the scope of this paper.

Force and Housing

A maximum force of nearly 1 500 men was employed on this work; for a year and seven months the number exceeded 1 000. Except for a small percentage of local labor, this force had to be imported and housed near the various shafts. Generally, men were housed in 28-man barracks, two men to a room, heated with stoves and lighted by electricity. The men were furnished with iron spring cots, mattresses, and blankets, which were regularly cleaned. They were fed in large mess halls, the food being ample and good.

Dry houses were built at each shaft, but, in general, were not used, probably owing to the dryness of the tunnels. Wash-houses were equipped with hot and cold water, shower baths, and facilities for washing clothes. Toilet facilities consisted of Kaustine units in buildings on the surface and portable Kaustine closets in the tunnel. An ample supply of wholesome water was piped from near-by springs or pumps. This supply was analyzed from time to time and was carefully guarded. Smaller houses were supplied for married men, and bungalows for superintendents and executives.

Due to the general unrest, the isolated location of the camps, and other conditions, the labor turn-over was high, frequently as much as 700% in a year.

Supplies and Transportation

Materials were brought by railroad to the Allaben Yard, or to a siding at Grand Gorge, and taken to the shafts by motor truck. The road from Shandaken to Lexington was of dirt—at some times fairly good, but, at others, nearly impassable. The contractor did considerable work on this road to keep it usable. In the winter, he kept the entire road along the tunnel line open, using snow plows attached to trucks or tractors. At times, it became necessary to shovel parts of the roads to keep them open. As an example of the amount of material handled, 609 carloads of cement and 387 carloads of other freight were hauled from the Allaben Yard alone in 1922.

Costs

The following are the contract costs to the city: The entire tunnel, including the Venturi meter and sluice-gates, cost $12 292 411, at the rate of about

$128 per ft., or $679 000 per mile. The surface work at the intake, including the superstructure and sluice-gates, cost $382 444. The sub-surface work, including the Venturi meter, cost $99 868. The outlet cost $132 448.

The shafts, excluding the intake, cost $367.52 per lin. ft., as follows:

Surface work	9%	=	$33.08—
Sinking	79.97%	=	293.91—
Lining	10.87%	=	39.95—
Grouting	0.16%	=	0.59+
Total		=	$367.52

The entire length of tunnel alone cost $109.21 per lin. ft., made up as follows:

Driving	65.71%	=	$71.76+
Timbering	2.51%	=	2.74+
Dry packing	0.47%	=	0.51+
Concrete lining	28.15%	=	30.74+
Grouting	3.16%	=	3.45+
Total		=	$109.21

The normal unsupported type of tunnel cost per linear foot:

To drive	$66.91
With temporary timber	72.63
To drive and line with concrete	96.30
With temporary timber	102.02
To drive and line with concrete and grout	98.09
With temporary timber	103.81

The normal supported type of tunnel cost per linear foot:

To drive	$77.05
With permanent timber	82.55
With permanent timber and concrete lining	115.31
With permanent timber, concrete lining and grout	122.22

Conclusions

From the experience gained on this work, it seems desirable that in tunnels of this size the shafts should be much farther apart. As will be noted from Fig. 5, consistent progress was maintained from Shaft 1 toward Shaft 3, even when the distance was greater than 2½ miles. This would indicate that shafts might be placed from 5 to 6 miles apart.

Progress in tunnel driving was limited by the removal of muck from the heading. A somewhat larger section permitting the use of an air-operated shovel might have been constructed more quickly and cheaply. To a considerable extent the progress of driving and concreting depended on the condition of the single track. Derailed trains caused by poor track retarded progress greatly. Well-kept double track might be economically sound for a long tunnel.

THE EYE-BAR CABLE SUSPENSION BRIDGE AT FLORIANOPOLIS, BRAZIL*

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Synopsis

The recently completed Florianopolis Suspension Bridge, with a main span of 1 113 ft. 9 in., is the longest span bridge in South America and the longest eye-bar suspension span in the world. The bridge was constructed for the Brazilian State of Santa Catharina, and spans the waters of a strait of the Atlantic Ocean. It is to carry a highway, electric railway, and water-supply to Florianopolis, the island capital of that State.

The structure is of interest to bridge engineers as the first executed example of a new form of suspension stiffening construction, whereby greatly increased rigidity and economy of material are secured simultaneously. The distinctive feature of this construction is the utilization of the cable to replace a part of the top chord of the stiffening truss, and the consequent change from the conventional parallel chord truss to a stiffening truss of more effective outline.

Another departure from customary practice is the use of rocker towers, yielding advantages in economy of material. The Florianopolis Bridge is the first suspension bridge in the Americas to be built with rocker towers.

The bridge is of further interest to engineers as the first application of an important new structural material. Instead of wire for the cables, eye-bars are used; and these are made of the newly developed, high-tension, heat-treated carbon steel, having a yield point exceeding 75 000 lb. per sq. in., and intended to be used with a working stress of 50 000 lb. per sq. in. The bid price for this type reduced the total cost to the lowest estimated cost with wire cables.

Originally the suspension type, of conventional design, was adopted for this bridge in economic competition with cantilever designs. Subsequent modifications yielded further economies in favor of the suspension type.

In the foundation work for the main piers, novel construction methods were devised to overcome unusual difficulties. The concrete anchorages are of special design, U-form in plan, for maximum efficiency. One anchorage is founded on rock, the other on piles.

Finally, an entirely new method was developed for the erection of the eye-bar cable and suspension span stiffening trusses, using an overhead

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trolley, thus eliminating wooden falsework and working platforms. The entire field operations extending over a period of one year were conducted without loss of human life.

This paper describes the methods by which the bridge was designed, fabricated, transported, and successfully erected.

History of the Project

The City of Florianopolis is the capital of Santa Catharina (Fig. 1), one of the twenty-one States of Brazil. The site is on the Island of Santa Catharina, separated from the mainland by a narrow strait. The city lies on the strait side of the island about midway from the ends. There, because of peninsulas extending out from both the mainland and island, the strait is only about 1 500 ft. wide. The water is too shallow for large vessels so that most ocean-going ships anchor at the northern end of the island, passengers and cargo being transferred to the city by tugs and lighters.

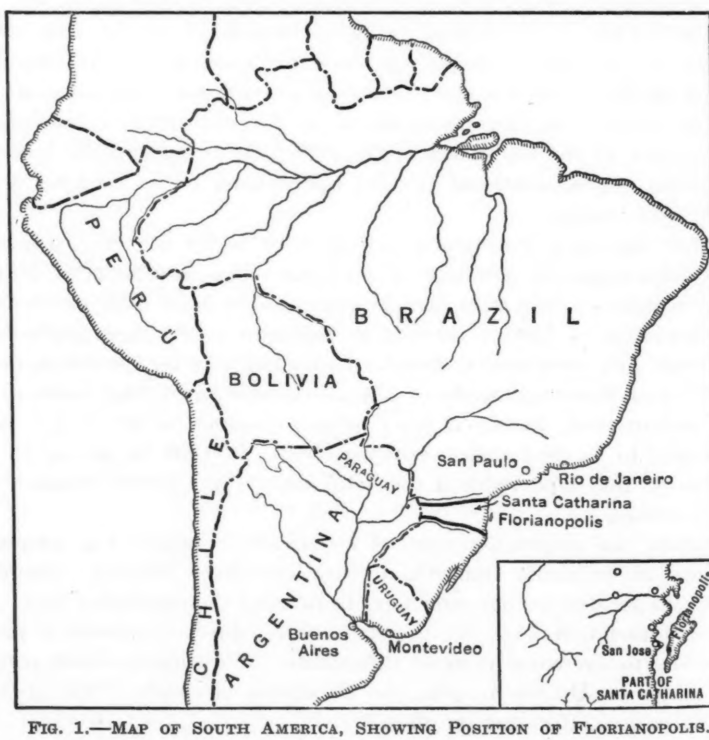

Fig. 1.—Map of South America, Showing Position of Florianopolis.

A few years ago the late Governor of Santa Catharina, Señor Hercilio Luz, conceived the idea of developing the Port of Florianopolis and the whole east central portion of the State of Santa Catharina in order to provide a means of outlet for the products of the interior part of the State

direct to ocean-going steamships for shipment to the United States and Europe. The entire project had a fourfold objective:

First.—To improve the harbor at the northern end of the island.

Second.—To build an electric car line about 12 miles long from the new port to the City of Florianopolis.

Third.—To construct a bridge connecting the island with the mainland.

Fourth.—To build about 100 miles of railroad direct into the interior connecting with the railroad extending from São Paulo, Brazil, to Montevideo, Uruguay.

Prior to 1920, various competitive designs had been submitted by American and European contractors and engineers for the proposed bridge at Florianopolis. Finally, the general contract was awarded to Byington and Sundstrom, Contracting Engineers, of São Paulo, Brazil, on the basis of a cantilever design. The bids on this design exceeded the appropriation and the general contractors wrote to H. D. Robinson, M. Am. Soc. C. E., about the advisability of preparing an alternative suspension design. Mr. Robinson in collaboration with one of the writers, Mr. Steinman, prepared a preliminary wire cable design.

In 1920, A. Y. Sundstrom, M. Am. Soc. C. E., came to New York, N. Y., and, acting for his firm, engaged Messrs. Robinson and Steinman to develop the suspension design for the Florianopolis Bridge. At the same time, he arranged with the American Bridge Company, through the United States Steel Products Company, to revise the cantilever design, and to submit price quotations on both types. On the basis of the respective unit prices quoted by the Steel Company, the suspension design lost out by a small margin. Consequently, when they secured from the State of Santa Catharina the general contract for the construction of the bridge, Byington and Sundstrom adopted the revised cantilever design, Fig. 2 (*c*). Before any work was commenced, the entire project was suspended by the failure of the New York banking firm that held the bridge funds, as proceeds of a bond issue.

In 1922 interest was revived by the availability of new funds, and the question of suspension *versus* cantilever design was re-opened by Byington and Sundstrom. The designs were slightly revised and new price quotations were secured from steel firms.

During this two-year interval the American Bridge Company had independently investigated several suspension designs and its engineers had reached the conclusion that this type was more economical than the cantilever. This was borne out when the bids were received as the suspension design was found to be more economical by a substantial margin, and, therefore, was adopted by the general contractors.

In fulfillment of the previous obligation, Byington and Sundstrom entered into a new contract with the United States Steel Products Company to furnish the fabricated steel, after satisfactory adjustments had been made in the unit prices. The general contractors, with the professional assistance

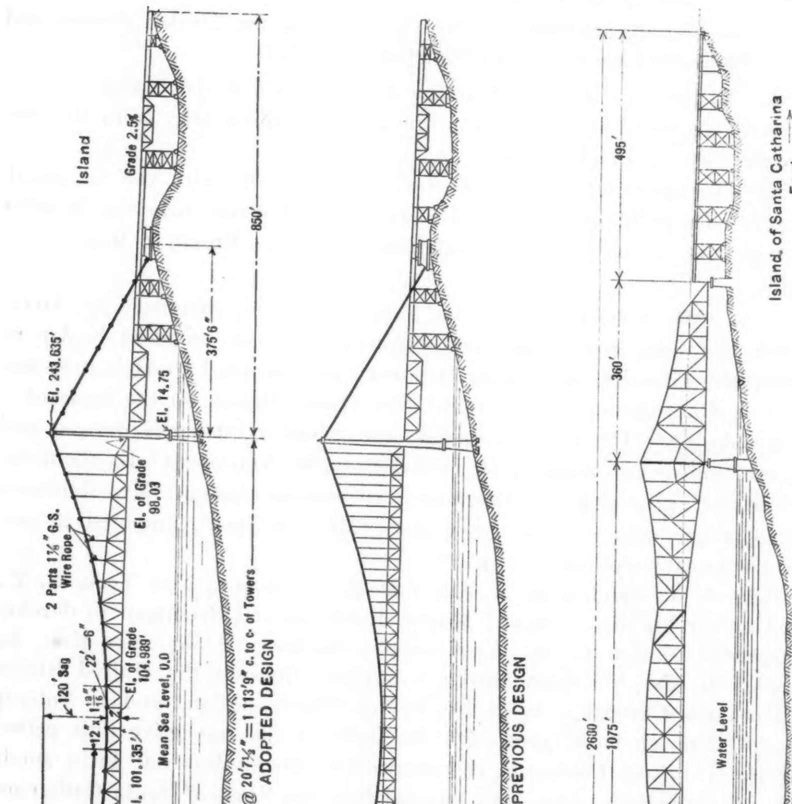

FIG. 2.—COMPARISON OF DESIGNS OF FLORIANOPOLIS BRIDGE.

of Messrs. Gross, Robinson, and Steinman, planned to handle the entire erection.

Messrs. Robinson and Steinman then proceeded with the completion of the plans. Alternative designs were made for the cables, using parallel wire and twisted wire-rope strands, respectively. Fig. 2 (*b*) shows the latter design. Bids were secured on both materials from steel wire firms in the United States and abroad, the lowest quotation being received from an English firm.

As Mr. Sundstrom was about to cable acceptance, one of the writers, Mr. Grove, of the American Bridge Company, recommended the use of its new high-tension, heat-treated eye-bars as a substitute for wire cables. The United States Steel Products Company supported this recommendation by a proposal to furnish and erect the eye-bars for a sum that proved to be less than the general contractor's estimate for the wire cables in place. Upon satisfactory price adjustment, the new material was accepted, and a sub-contract for its fabrication and erection was added to the previous contract for furnishing the other fabricated steel. To save duplication of erection equipment and organization, the remainder of the steel erection (towers, stiffening trusses, approach viaducts, etc.) was then added to the contract, relieving the general contractors of the entire superstructure.

The adoption of the eye-bars as a substitute for the wire cables suggested to Messrs. Robinson and Steinman a change of design from the parallel-chord type of stiffening truss to some form that would utilize the availability of the eye-bar cables for greater effciency of the stiffening construction. After several studies made with this object in view, a curved chord design utilizing the cable to replace the middle half of the top chord, Fig. 2 (*a*), was selected. Upon the evidence of a material economy in its favor, the change in design was approved by Mr. Sundstrom and was accepted by the Steel Company. A photograph of the completed structure is shown in Fig. 3.

Location of the Bridge

The location of the crossing is shown in Figs. 4 and 5. The east, or island, end of the bridge (background, Fig. 4) was determined by a rocky promontory pointing toward the continent, and by a high rocky knoll a short distance back affording an ideal anchorage site. The west, or continent, end of the bridge (foreground, Fig. 4) was at a point where soundings indicated shallow depth for main pier foundations, and where the steep abutting topography provided a practical approach for the electric railway connection. To facilitate this connection, a curve was introduced in the west end of the approach viaduct.

The Government contract specified a main span of 340 m. for either a cantilever or a suspension bridge. This span length was based on the preliminary soundings and layouts, and was retained (with slight changes for English units and uniform panel lengths) in the final design.

Considerations of the profile, anchorage sites, highway crossings, and duplication of spans determined the span lengths and arrangement of the towers and spans of the two viaduct approaches.

New Type of Stiffening Truss

The form of stiffening construction adopted for the Florianopolis Bridge is an innovation. A comparison of the adopted design with other conventional layouts which it superseded (Fig. 2), shows the distinctive characteristics of the new type.

Utilization of the Cable as Truss Chord.—In the central portion of the conventional form of suspension construction, the cable, sustaining the dominant tensile stress, closely follows the upper chord of the stiffening truss, which has compression for its governing stress. Such juxtaposition of two principal members carrying opposing stresses represents a waste of material or, rather, a neglected opportunity for economizing. By combining the two opposing structural elements, one member is made to take the place of two; the result is a subtraction of stresses instead of an addition of sections. This effects a partial neutralization of the maximum tension in the middle portion of the cable, and the omission of a corresponding portion of the upper chord of the truss.

This utilization of the cable as the upper chord of the stiffening truss should preferably be limited to the central half of the span. To extend this construction to the ends of the span would not be economical, because the saving of the top chord in the outer quarters would be offset by the increase in the length of the web members in a region where the stiffening truss has its maximum shears. Moreover, beyond the quarter-points there would be an addition instead of a subtraction of stresses, since the condition of loading that produces maximum tension in those top chords is one that produces nearly maximum tension in the cable.

Variation of Truss Profile.—Another neglected opportunity for increasing economy and efficiency in the conventional form of suspension construction is in the use of parallel-chord stiffening trusses. For maximum economy the truss should have a profile conforming to the variation of maximum bending moments along the span, a principle that is recognized in designing other structures, as simple trusses, cantilevers, continuous trusses, and arches. Since the economic depth at any section is a function of the governing bending moment, a truss should have its greatest depth at the points of greatest bending moment, and should be made shallow where the bending moments are comparatively small.

In a suspension-bridge stiffening truss, the greatest bending moments occur near the quarter-points of the span; consequently, the economic profile of a stiffening truss is one having maximum depth near the quarter-points and mimimum depth at mid-span and at the ends. This conclusion is strengthened by the fact that the shears in a stiffening truss are a minimum near the quarter-points and attain maximum values at the middle and ends of the span. Thus, a truss profile with maximum depth near the quarter-points

FIG. 3.—VIEW OF FLORIANOPOLIS BRIDGE, COMPLETED, 1926.

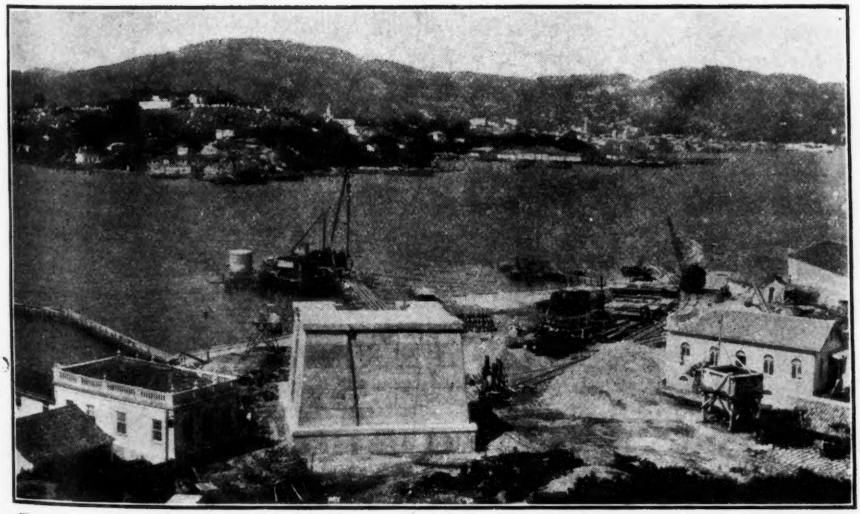

FIG. 4.—VIEW OF CROSSING DURING CONSTRUCTION OF MAIN PIERS, FLORIANOPOLIS BRIDGE.

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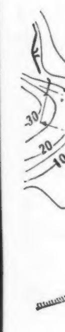

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also gives economy in web members since it provides the shallowest depth in the regions where the web stresses are greatest. Such a profile yields the additional advantage of greater uniformity of required chord sections throughout the span as a wide range of variation usually involves a waste of material in those chord members requiring minimum sections.

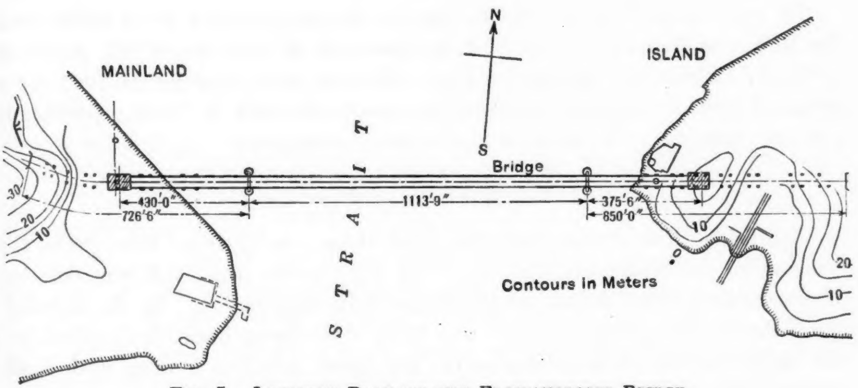


FIG. 5.—LOCATION PLAN OF THE FLORIANOPOLIS BRIDGE.

Another consideration governing truss depth is that of efficiency in reducing deflections. The most serious deflection of a stiffening truss, as measured by the resulting deflection gradients, is produced under the condition of live load covering approximately one-half the span. Half-span loading produces a downward deflection of the loaded segment and a smaller upward deflection of the unloaded segment, with a maximum deflection gradient at the loaded end. The magnitude of this deformation depends on the truss depth at and near the quarter-points. Calculations show that to limit the deflection gradient to 1%, a truss depth of about one-forty-fifth of the span is required. A parallel chord stiffening truss of such depth (as illustrated by the Williamsburgh Bridge, New York, N. Y.) would render the structure unsightly. To secure the requisite stiffness without resorting to a stiffening truss of clumsy proportions, it is necessary to depart from the parallel-chord type and to adopt an outline providing the extreme depth only where it is needed, namely, in the vicinity of the quarter-points of the span.

Two Features Combined.—From the foregoing considerations the two logical means for improving conventional suspension design for increased economy and efficiency are:

1.—Utilization of the cable to replace a portion of the top chord of the stiffening truss (preferably limited to the middle half of the span).

2.—Variation of the truss profile to give maximum depth near the quarter-points of the span.

Fortunately, compliance with the first of these requirements automatically helps with the second. The result is the form of suspension construction adopted for the Florianopolis Bridge.

Originality of the Type

Although the Florianopolis Bridge is the first executed example of this new type of suspension construction, the idea appears to have been anticipated by others. The following prior proposals of designs embody the same or similar ideas.

In 1895, Landsberg described a similar design proposed for a bridge over the Rhine, at Bonn, Germany.* L. S. Moisseiff, M. Am. Soc. C. E., reported† a design embodying similar features which he had prepared in 1907 for a proposed 1 200-ft. span, with 600-ft. side spans. In 1911, R. Sontag published‡ a paper suggesting similar ideas for suspension designs.

The Revision of Design

The bridge had already been designed along conventional lines, when the decision to substitute eye-bar cables for wire cables prompted consideration of the revised truss design and facilitated its application. In the parallel-chord design, the stiffening truss was 25 ft. deep throughout; the revised lay-out, utilizing part of the chain as the top chord, provides a truss with depth varying from 22.5 to 42.5 ft.

In the first plans for the new design, the upper chord in the outer quarters of the span was made curved so as to produce an effect of symmetry about the quarter-points. Straight chords were substituted, however, at the decision of the purchaser, in order to minimize fabrication costs. Minimum cost was the outstanding requirement governing the entire design. The question of relative appearance of straight and curved chords in the outer quarters of the span is a matter of individual preference.

Although the revision from the conventional design was prompted and facilitated by the adoption of eye-bar cables for the principal suspension elements, the new form of stiffening construction can also be used in conjunction with wire cables.§ Approved details of the necessary connections between truss members and wire cables have been developed by Messrs. Robinson and Steinman for designs of this type.‖

Economy of the Adopted Design

Comparative cost estimates of the two designs shown in Fig. 1 (*a*) and (*b*) demonstrate a material saving in favor of the adopted design. In addition to the major elements of economy inherent in the salient features of the new form of construction, a number of incidental savings arise from the change in design:

* Described in *Zentralblatt der Bauverwaltung* and proposed by the Maschinenfabrik Esslingen; see discussion in *Die Bautechnik*, March 13, 1925; also *Der Bauingenieur*, No. 35, December, 1925.

† *Engineering News-Record*, November 27, 1924.

‡ *Eisenbau*, July, 1911.

§ Used in the suspension design by Messrs. Robinson and Steinman for the Sydney Harbor Bridge.

‖ *Engineering and Contracting*, June 24, 1925, pp. 1377–1386; *Die Bautechnik*, 1925, No. 33, pp. 451–452, and 1925, No. 44, p. 630.

1.—Saving the material represented by the middle half of the top chord of each stiffening truss.

2.—A general saving in the remaining chord material resulting from the use of an economic truss profile conforming to the variation of bending moments along the span; and, in particular, a material reduction in the maximum chord sections, near the quarter-points.

3.—A saving in details and in minimum sections resulting from the greater uniformity of required chord sections throughout the span.

4.—A saving in web material on account of the reduced truss depth in the regions of maximum shear.

5.—The omission, in the middle half of the span, of the sub-verticals previously required to shorten the compression-chord members, now replaced by tension members.

6.—The omission of the intermediate top laterals previously required to stay these shortened compression chord members in a horizontal plane.

7.—As a result of these savings, a reduction of about one-third in the total weight of the stiffening truss, and a consequent further saving in all parts affected by the dead load of the truss.

8.—A saving in cable sections resulting from the reduced dead load of the truss and from the consequent reduced dead load of the cable.

9.—The omission of the suspenders in the middle half of the span and a reduction in length of the remaining suspenders.

10.—A combined saving in the towers resulting in part from the reduction (6.5 ft. in the case of the Florianopolis Bridge) in the total height in consequence of the reduced distance between the cable and lower chord at the center.

11.—A material saving in the anchorages resulting from the reduced dead load of the truss and cable and the reduced elevation of the back-stays.

In the case of the Florianopolis Bridge, much of this economy was not capitalized but was turned back into the structure in the form of a general reduction of unit stresses to provide a greater margin of safety for future load increases. Thus, the design stress for the stiffening truss was lowered from 20 000 to 18 500 lb. per sq. in. (actually 14 500 lb. as calculated by the exact method); and the unit stress in the cable was reduced from 50 000 to 46 500 lb. per sq. in.

Designs have been proposed, in the past, in which the cable would be utilized as the top chord of an overhead bracing system. G. Lindenthal, M. Am. Soc. C. E., advocated for the Manhattan Bridge and for the Quebec Bridge, eye-bar cable designs with bracing systems having the maximum depth near the quarter-points of the span. In those designs, however, the stiffening system would have to leave the roadway level to follow the line of the cable in the outer quarters of the span.

The design of the Florianopolis Bridge secures the desired advantages while retaining a stiffening truss at the roadway level from tower to tower. The use of an overhead trussing system (departing from the roadway level) would necessitate separate wind chords for lateral stiffening.

Gain in Rigidity

Although the governing consideration in the change of design was that of economy, the change yielded greatly increased rigidity (300%) as an incidental advantage.

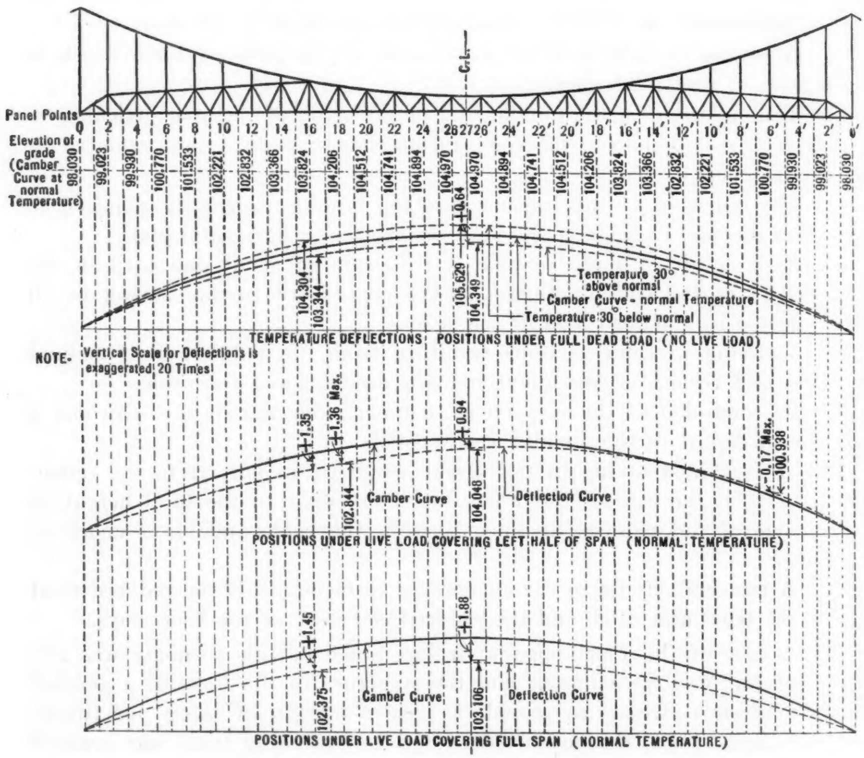


Fig. 6.—Calculated Deflection Graphs for the Florianopolis Bridge.

According to the deflection graphs calculated for the adopted design (Fig. 6), the maximum deflection under full-span loading is only 1.88 ft., or $\frac{1}{592}$ of the span; and the maximum deflection under half-span loading is only 1.36 ft., or $\frac{1}{820}$ of the span, with uplift in the unloaded half practically eliminated. The actual deflections will be about 30% less than these calculated values, since the deflection calculations were based on the elastic, or "approximate", method and since no allowance was made for the stiffening effect of details. These deflections are approximately one-fourth of the corresponding values for the previous (conventional) design. As the Florianopolis Bridge is designed to carry a railway as its principal element of live load, this reduction of the governing deflections is of practical significance.

An increase in rigity of about 25% may be attributed to the substitution of eye-bars for wire cables; the remaining 275% increase is the direct consequence of the new form of stiffening construction.

The following elements of the new design contribute to this increase in rigidity:

1.—The revised truss profile is more efficient in resisting deflections, since it provides a greater average depth, with maximum depth in the regions of greatest bending moment.

2.—The depth at the quarter-points has been made nearly twice as great as in the previous design, and the stiffness in the vicinity of the quarter-points is the principal factor in determining the rigidity of a suspension bridge under the critical condition of half-span loading.

3.—The functioning of the full section of the cable as the top chord of the stiffening truss in the middle half of the span greatly increases the moment of inertia in that part of the span.

4.—The fact that the live load introduces tension in the middle half of the top chord (by virtue of its forming part of the cable) further reduces mid-span deflections.

As a result of these various factors, the change of design yields greater stiffness with less material in the structure. In approximate figures, the design is four times as rigid with only two-thirds as much material in the stiffening truss. Thus, greater efficiency has been secured through a more scientific design of the suspension stiffening system.

In addition to the marked increase in vertical rigidity yielded by the new design, the lateral stiffness is improved by the large cable sections functioning as wind chords in the middle half of the span; and ideal longitudinal rigidity is secured by the direct connection of the truss to the cables. Longitudinal or braking forces are carried directly into the cable.

The net result of the change in design as applied to the Florianopolis Bridge is a reduction in cost (through actual saving in material), an increase in safety and longevity (through lowered unit stresses), and an increase in efficiency (as measured by resistance to deflections).

Rigidity in Suspension Bridge Design

There are differences of opinion among bridge engineers as to the desirability of rigidity in suspension designs. Those who advocate maximum stiffness as a desideratum are applying to suspension systems considerations borrowed from other types of bridge construction. In those other types, rigidity, strength, safety, and longevity are intimately related.

In the suspension type, on the other hand, strength, safety, and longevity are not dependent on the degree of rigidity. A suspension bridge has greater safety and longevity than other types, even when its deflections are many times greater; and its strength is not measured by its rigidity.

Long-span suspension bridges of recent design or construction have been intentionally proportioned to permit maximum deflections of as much as 15 or 20 ft., without impairing the safety or permanence of the structure. A

certain amount of flexibility is an advantage in a structure, providing, as it does, resilience to resist the effects of impact and shock. Nevertheless, for practical reasons, proper methods must be available for restricting or minimizing suspension-bridge deflections. One of these reasons is the requirements of railway traffic. For such loading, the permissible deflection gradients are limited; and the "traveling wave", involving the upward deflection of the unloaded part of the span, is cited as an objection. A design of the Florianopolis type permits the deflection gradients to be restricted adequately, and the "traveling wave" to be eliminated.

Another reason is a prejudice among laymen and others, against the suspension type on the score of its flexibility. Adverse impressions derived from flimsy, improperly designed, suspension spans are that the suspension type is necessarily light and shaky. To meet these conditions, it is most desirable to be able to limit and control the deflections of suspension designs.

Suspension Bridges Can Be Designed to Have Any Desired Degree of Rigidity

In the Sydney, Australia, Harbor Bridge competition, the suspension type was officially excluded on the score of alleged excessive flexibility; the engineer in charge declared that the deflection of the 1 600-ft. span under the heavy railroad traffic would amount to 11 ft. in a suspension design. As a challenge to this objection, a design submitted by Messrs. Robinson and Steinman embodied an extension of the principles of the Florianopolis type, the stiffening trusses being extended as a continuous structure through the towers to the anchorages. The calculated maximum deflection under the full live load of 12 000 lb. per lin. ft. was only 1.28 ft.; and the calculated maximum deflection under half-span loading was only 0.83 ft., or $\frac{1}{1\,930}$ of the span, with only an imperceptible uplift (a fraction of an inch) in the unloaded half of the span.

The Florianopolis type, by supplying great depth and stiffness at the quarter-points, minimizes the critical half-span deflections. The tendency toward upward deflection in the unloaded half-span is thus reduced and its value just about balances the downward tendency from the deflection of the span as a whole; consequently, there is no upward movement, or practically none, of the unloaded half of the span. In this manner, the "traveling wave" under an advancing train load may be completely eliminated with such design.

With the parallel-chord stiffening truss, flexibility is reducible by deepening the stiffening truss, usually at a sacrifice of both economy and appearance. With the Florianopolis type of construction, greatly increased rigidity may be secured without resorting to clumsy proportions and with a reduction, instead of an increase, in the weight of steel required.

Esthetic Considerations

Discussions of comparative æsthetic values in structural design are generally unsatisfying on account of the existing differences of individual taste.

Any attempt to compare the Florianopolis design with the conventional suspension design must suffer from that limitation. At first comparison with the conventional type, the new design is under a certain disadvantage on account of the subconscious prejudice against novelty of form in favor of that which is familiar through custom.

Nevertheless, the new outline has certain æsthetic values to commend it:

1.—It produces an impression of sturdiness and strength in contrast with the apparent lightness and inadequacy of the usual suspension design.

2.—It relieves the effect of a long straight stiffening truss by introducing a curvilinear variation in depth of truss; this variation is particularly pleasing when the top chord in the outer quarters of the span is also made curved in symmetry.

3.—It contributes certain pleasing values through the more harmonious adjustment of chord outline to curve of cable.

4.—It produces a sense of a stronger functional relationship between truss and cable.

5.—The reduction of lines and members to a minimum enhances the effect of sturdy simplicity.

Finally, the new design has a certain æsthetic value on account of its obvious efficiency. Conflicting and redundant lines have been eliminated, and adjustment of form to function has been emphasized. The harmonious expression of efficiency and strength is, after all, the essence of structural æsthetics.

Heat-Treated Eye-Bars

In the Florianopolis Bridge a new material, in the form of high-tension, heat-treated, carbon-steel eye-bars, found its first application. This material, intended to be used with a working stress of 50 000 lb. per sq. in., was developed through experimental research by the American Bridge Company. It is furnished under guaranty of minimum elastic limit of 75 000 and minimum ultimate strength of 105 000 lb. per sq. in., and minimum elongation of 5% in 18 ft.

Heat treatment of steel has been known and used for some time, but it is only within the last decade that its application has been made to structural steel bridges. Carbon steel is the material used, but the amount of carbon is higher than in the ordinary structural grade with an ultimate strength of 55 000 to 65 000 lb. per sq. in.

After the steel is manufactured and rolled in the eye-bar sizes, the eye-bars are upset in the usual manner, except that the heads are made ⅛ in. thicker than the body of the bar. The bars are then placed in the heat-treating or annealing furnaces and subjected to temperatures necessary to produce elastic limits and ultimate strengths of the desired amounts. After quenching, the bars are re-heated and then cooled slowly. Each bar is treated separately.

On account of the nature of the work of heat-treating, the Steel Company felt obliged to place restrictions around the inspection. There were more than 400 bars on the bridge to be heat-treated separately, each operation taking from 4 to 5 hours. All bars were 12 in. wide and varied from $1\frac{3}{8}$ to 2 in. in thickness. The 12 by $1\frac{3}{8}$-in. bars were about 20 ft. long, all others being 40 ft., or more. Thus, all bars except the 20-ft. bars exceeded in length the limits for the testing machine. Cutting and re-heading of test pieces selected from bars exceeding 40 ft. in length meant that these bars would have to be re-treated as the heating necessary for re-heading the bars would destroy the effect of the previous heat treatment. It was decided therefore to test two short and ten long bars. The two additional short bars were added to the thirty-two short bars needed and the entire group of thirty-four were headed and heat-treated. The inspector then selected two of the finished bars for Test Pieces Nos. 1 and 2.

Enough additional material was ordered from the mill for two bars, 12 by $1\frac{13}{16}$ in.; two bars, 12 by $1\frac{7}{8}$ in.; two bars, 12 by $1\frac{15}{16}$ in.; and four bars, 12 by 2 in. Before the remainder of the bars were made, the inspector selected from the mill material ten bars of the sizes mentioned. These ten pieces were cut to lengths to fit the testing machine, headed and heat-treated for Test Bars Nos. 3 to 12, inclusive.

The policy of secrecy covering the material and processes of heat-treatment was unsatisfactory to the consulting engineers, but the Steel Company took the position that its contractual obligation was to furnish bars with stated properties, and that if the full-sized test bars satisfied these requirements, the processes giving these results were not open to the inspector but were to be considered as private. Under those circumstances, the consulting engineers declined to assume any responsibility for the strength and safety of the eye-bars furnished for the structure; and the matter was left with the understanding that the Steel Company assumed sole responsibility for that material.

In nine full-sized tests made during the period of experimental development, prior to the offering of the material for the Florianopolis Bridge, the physical properties ranged as shown in Table 1.

TABLE 1.—PROPERTIES OF HEAT-TREATED STEEL.

	Maximum.	Minimum.	Average.
Elastic limit, in pounds per square inch	89 000	78 500	83 920
Ultimate strength, in pounds per square inch	132 600	113 900	123 200
Percentage of elongation (in 10 ft.)	9.2	6.1	7.2
Percentage of contraction of area	39.2	16.5	24.6

Of the twelve full-sized tests made during the fabrication of the eye-bars for Florianopolis, only one eye-bar, No. 12, failed to meet all the specified requirements, passing the elastic limit and ultimate strength requirements,

but having an elongation of only 3.8 instead of 5 per cent. A new eye-bar, No. 13, was made up for "re-test" and passed the requirements with an elongation of 8.1 per cent. The complete physical properties for the thirteen test bars are given in Table 2.

It is possible to check the uniformity and reliability of this material by Brinell tests supplementing the strength tests. A comparison of the Brinell hardness numbers secured at different points of the finished eye-bar furnishes an index of the uniformity of treatment and physical qualities over the length of the bar.

TABLE 2.—Physical Properties of Full-Sized Test Eye-Bars, Florianopolis Bridge.
Tests Made During May, 1924, and January, 1925.

No.	Nominal section, in inches	Length.		Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Percentage of elongation.	Fracture.
		Feet.	Inches.				
1	12 by 1$\frac{3}{8}$	17	6	88 000	126 450	8.08 in 13 ft.	40% silky cup, 60% silky square
2	12 by 1$\frac{3}{8}$	17	6	96 830	137 900	6.43 in 13 ft.	Silky square
3	12 by 1$\frac{13}{16}$	39	0	90 100	132 600	6.86 in 18 ft.	100% silky square
4	12 by 1$\frac{13}{16}$	40	0	82 810	122 820	7.7 in 18 ft.	30% cup, 70% silky square
5	12 by 1$\frac{7}{8}$	40	0	85 710	124 600	7.4 in 18 ft.	Silky square
6	12 by 1$\frac{7}{8}$	40	0	86 740	127 160	6.5 in 18 ft.	Silky square
7	12 by 1$\frac{15}{16}$	40	0	83 340	120 300	7.2 in 18 ft.	Silky square
8	12 by 1$\frac{15}{16}$	40	0	78 180	115 970	7.6 in 18 ft.	Silky square
9	12 by 2	40	0	83 470	122 500	6.3 in 18 ft.	Silky square
10	12 by 2	40	0	79 440	114 660	6.6 in 18 ft.	Silky square
11	12 by 2	40	0	81 190	116 820	5.7 in 18 ft.	Silky square
12	12 by 2	40	0	82 780	116 720	3.8 in 18 ft.	Silky square
13	12 by 2	40	0	79 960	120 350	8.1 in 18 ft.	Silky square
Average				84 500	123 000	6.8 in 18 ft.	
Minimum requirements				75 000	105 000	5.0 in 18 ft.	

Use of Rocker Towers

The Florianopolis Bridge is believed to be the first American suspension bridge built with rocker towers. The only large bridges previously built with this feature are the Elizabeth Bridge at Budapest, Hungary (1903), and the bridge over the Rhine at Cologne, Germany (1915).

The evolution of suspension-bridge tower design has been marked by three successive types. First came the rigid form of tower, typified by the masonry towers of the Brooklyn Bridge. The principle of rigid-tower construction persisted for some time after steel replaced masonry, as illustrated in the towers of the Williamsburgh Bridge (New York City). When engineers realized that the rollers on the tower tops were usually inoperative and could not be relied on to provide the necessary movement of the cable saddles, the flexible type of tower was introduced. In that type, the saddles are fixed on the tops of the towers, and the horizontal movement necessitated by unbalanced cable pull is provided by flexure of the slender, fixed-base towers. First

used in the Manhattan Bridge (also in New York City), the flexible type of tower has been applied in the more recent structures, such as the Rondout (at Kingston, N. Y.), Bear Mountain (over the Hudson River), and Philadelphia-Camden Bridges. As a further step toward the reduction and simplification of tower stresses, the rocker type of tower has been developed.

The rocker type offers the most economical and scientific design for suspension bridge towers. It eliminates the bending stresses from unbalanced cable pull, thereby yielding a saving in tower material, and it obviates the difficulties of the necessary erection operation of pulling back the tops of the towers prior to stringing the cables.

In the case of the Florianopolis Bridge, the change was made from fixed bases after comparative estimates showed a net saving of about 20% in the weight of the towers in favor of the rocker type. There is a substantial reduction in the main sections by the elimination of the bending stresses. Another important advantage is the elimination of bending stress from the piers, permitting their size and reinforcement to be reduced.

Details of the Tower Design

As shown in Fig. 7, the towers are approximately 230 ft. high. The legs are battered, from a top width of 33 ft. 6 in. to a width of 55 ft. 6 in. at the base. The width at the top corresponds to the spacing, center to center, of trusses, the cables hanging in vertical planes. This form of tower design, introduced into suspension bridge construction by Mr. Robinson, has for its chief advantage the facility of running the truss and roadway construction through the tower portal without interference from the tower legs; an additional advantage is the increased transverse stability, of particular importance for narrow bridges.

The two legs of the Florianopolis tower design are braced together with rigid diagonal and transverse bracing members, the latter including a transverse distributing girder at the tower top, a transverse reaction girder (Fig. 8) supporting the stiffening truss and approach truss bearings, and a transverse portal girder (Fig. 9) immediately above the stiffening truss portal.

The reaction girder (Fig. 8) has a box section and supports the roller expansion bearings of the approach span and the pin-connected rocker supports of the main span stiffening truss. These rockers take care of the vertical reactions (both positive and negative) while permitting the necessary longitudinal expansion movements.

Each tower leg or column is made up of a double box section (Fig. 7), having a maximum longitudinal width of 8 ft. and a constant transverse width of 3 ft. 6½ in. The 8-ft. width tapers to 4 ft. 6 in. at the top, and to 5 ft. 0 in. at the base. Transverse stiffening diaphragms are provided at intervals, two in each column section.

The tower is designed for a maximum horizontal component of 3 860 000 lb. in each cable, resulting in a maximum vertical reaction of 3 790 000 lb. per column; this is supplemented by the vertical reactions of the stiffening truss (240 000 lb.) and approach span (300 000 lb.), which reactions, however,

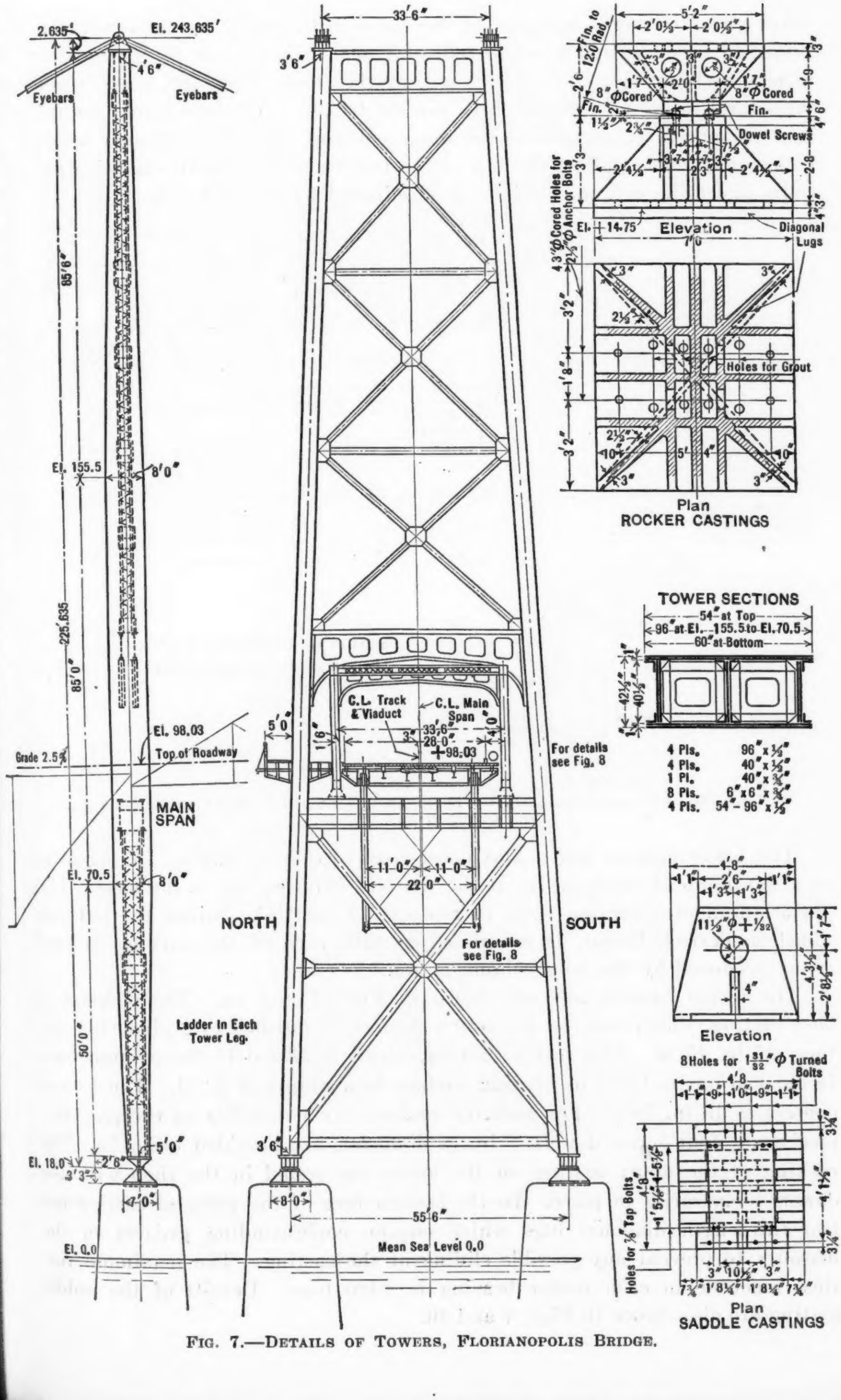


FIG. 7.—DETAILS OF TOWERS, FLORIANOPOLIS BRIDGE.

affect only the lower sections of the tower. In addition, the tower top is subjected to a maximum lateral pull of 75 000 lb. per tower from wind forces transmitted through the cables; and, at the lower level, the tower receives the lateral reactions (190 000 lb.) from the trusses. All these forces are taken into account in proportioning the tower columns and the transverse bracing. Unbalanced cable pull at the top of the towers, which would otherwise materially affect the column sections, is eliminated by the rocker feature.

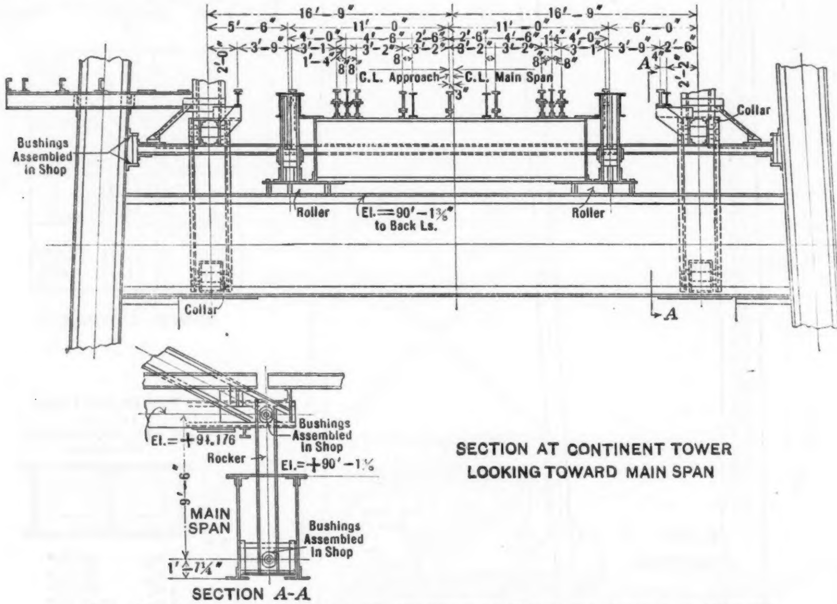


FIG. 8.—ASSEMBLY DIAGRAM OF TRANSVERSE REACTION GIRDER IN TOWERS OF FLORIANOPOLIS BRIDGE.

The tower column has a maximum cross-section of 370 sq. in., tapering to a minimum of 285.5 sq. in. at the top and 297.5 sq. in. at the base. This variation in cross-section (and in moment of inertia), similar to that provided in derrick booms, is calculated to take care of the varying flexural stress produced by the long column action.

The rocker base details are shown in Figs. 7 and 10. The pedestal or base casting, which rests on the concrete piers, is finished to a plane top surface, 27 by 45 in. The rocker casting, which is affixed to the column base, is finished on its lower or bearing surface to a radius of 12 ft. The line of contact is 45 in. long. For security against any possibility of creeping displacement, four screw dowels, 3 in. in diameter, are provided (Fig. 7). The rocking of the upper casting on the lower was tested in the shop with the dowels temporarily in place. In the bottom face of the pedestal casting are two full-length diagonal lugs which engage corresponding grooves in the masonry to prevent any possible sliding of the casting. The maximum vertical reaction on each rocker bearing is 2 420 tons. Details of the saddle casting are also shown in Figs. 7 and 10.

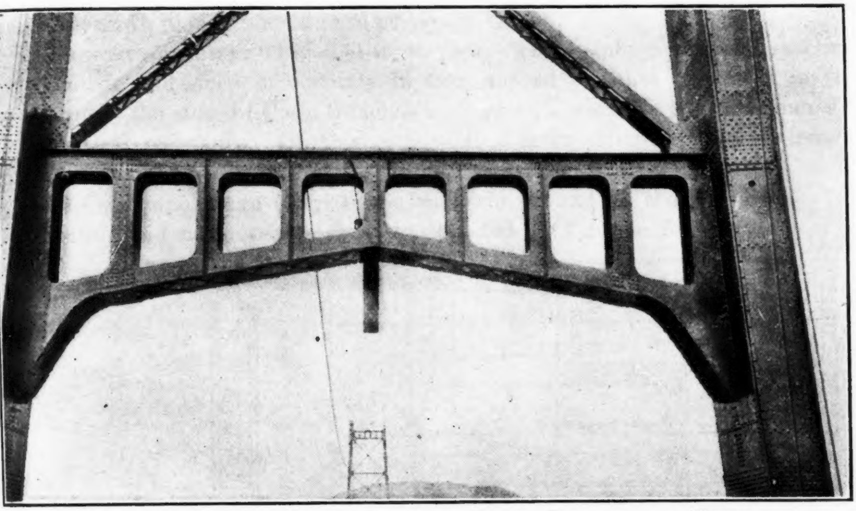


FIG. 9.—TRANSVERSE PORTAL GIRDER IN TOWER, FLORIANOPOLIS BRIDGE.

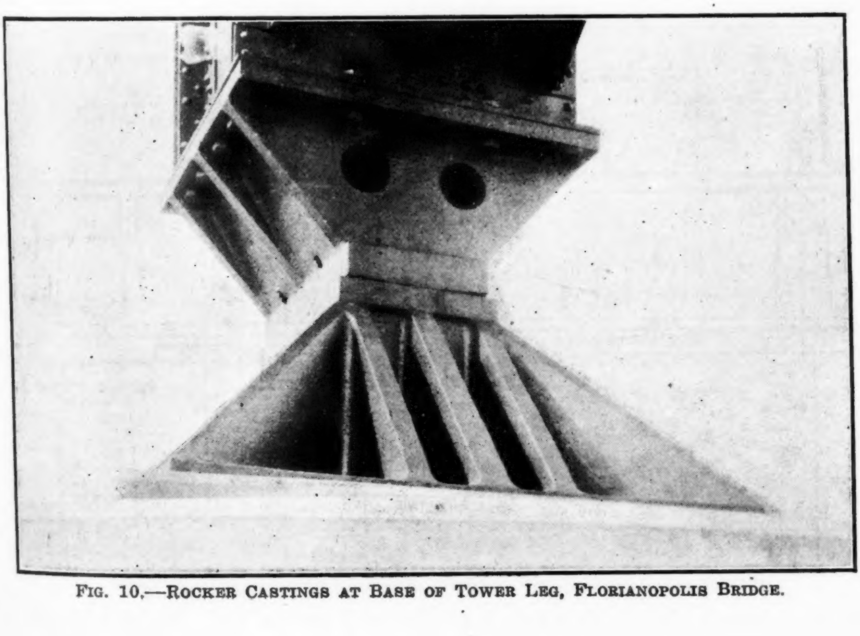


FIG. 10.—ROCKER CASTINGS AT BASE OF TOWER LEG, FLORIANOPOLIS BRIDGE.

Design and Construction of the Anchorages

The design of the island anchorage is shown in Fig. 11. Both anchorages are **U**-form in plan, for maximum efficiency.

The east anchorage (Fig. 11) is on rock. The anchor cables and reaction girders are embedded in concrete in two stepped trenches excavated in the solid rock; the sides of these trenches were given a negative batter to increase the vertical resistance. On this construction is superimposed the buttressed concrete anchorage structure.

The west anchorage (Fig. 12) is on lower ground on the mainland. The excavation did not reveal rock as anticipated, and a pile foundation had to

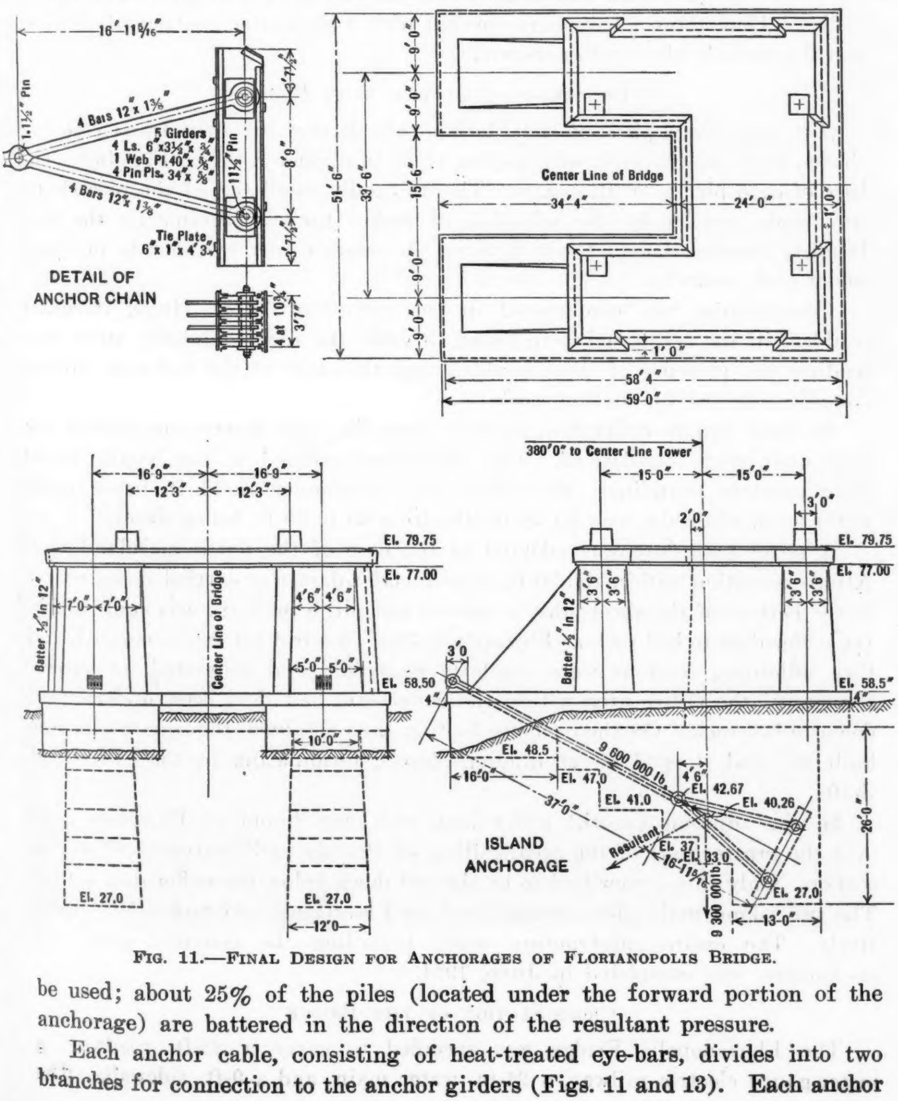


FIG. 11.—FINAL DESIGN FOR ANCHORAGES OF FLORIANOPOLIS BRIDGE.

be used; about 25% of the piles (located under the forward portion of the anchorage) are battered in the direction of the resultant pressure.

Each anchor cable, consisting of heat-treated eye-bars, divides into two branches for connection to the anchor girders (Figs. 11 and 13). Each anchor

girder, 16 ft. long, consists of five built-up girders reinforced with pin-plates and connected together by tie-plates. At the Y-point of each anchor chain, a built-up pin seat was provided to hold the pin in accurate position during the placing of the concrete around the anchor girder and the connecting eye-bars; these temporary pin seats were burned off after the girders and the lower eye-bars were securely concreted. (See Fig. 13.) Above these points, the anchor cables were boxed in during the completion of the concreting in order to prevent adhesion of the concrete before full dead load strain was in the cables. When the anchorage erection was completed, the boxing was removed; the tunnels were left around each anchor cable until the steel superstructure was erected and the full dead load stress was in the cables. Then the eye-bars were covered with a protective coating of minwax and the tunnels filled with concrete.

Construction of the Main Piers

The four main piers of the Florianopolis Bridge are cylindrical concrete shafts, 16 ft. in diameter, with coping 17 ft. in diameter. (See Fig. 16.) The base of each pier is 30 ft. square. The unusually small size of the pier shafts was made possible by the adoption of rocker towers, eliminating the pier bending stresses due to tower flexure. A separate pier cylinder is provided under each tower leg.

Construction was commenced in the spring of 1923. Huge, irregular boulders in the ocean bed were found to make the rock soundings quite misleading, the presence of these boulders and the slope of the bed-rock surface complicating the foundation work.

An open square coffer-dam of steel sheet-piles was driven for each of the four main pier foundations. The sheet-piles, ordered to the length found from previous soundings, went down to a maximum of 38 ft. below mean water level, while the rock lay at depths from 30 to 60 ft. below water.

A novel procedure was adopted in the case of the north mainland pier. After excavation within the 30-ft. square coffer-dam was carried down nearly to the bottom of the sheet-piles, a central test pit, 8 by 8 ft., was sunk within the coffer-dam to bed-rock at Elevation —59. This test pit was concreted, and then adjoining sections were similarly excavated and concreted, in succession, until the entire area within the coffer-dam had thus been underpinned down to bed-rock. On the concrete footing thus obtained, the square pier was built up, and stepped off at different levels, terminating in the cylindrical shaft.

In the adjacent (south) coffer-dam, rock was found at Elevation —32 over the greater part of the area, falling off sharply to Elevation —60 at one corner. Only this corner had to be sheeted down below the coffer-dam piling. The two island main piers reached rock at Elevations —37 and —50, respectively. The entire substructure work, including the concrete piers and anchorages, was completed in June, 1924.

Cross-Section of the Bridge

The Florianopolis Bridge was specified to carry a 28-ft. roadway, a meter-gauge electric railway, a 24-in. water main, and a 9-ft. sidewalk. The

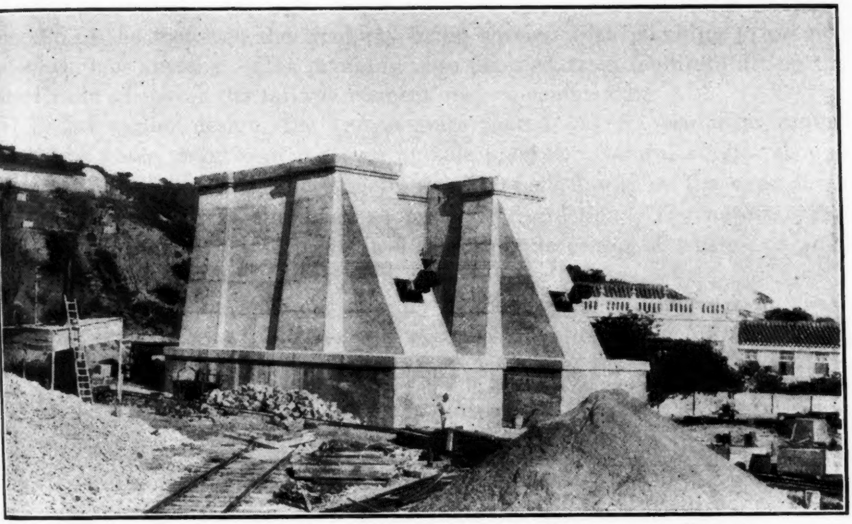


FIG. 12.—CONTINENT ANCHORAGE, COMPLETED, FLORIANOPOLIS BRIDGE.

FIG. 13.—ANCHOR GIRDERS AND EYE-BARS IN PLACE BEFORE CONCRETING, FLORIANOPOLIS BRIDGE.

arrangement of cross-section finally adopted is shown in Fig. 14 (*a*); that of the approach viaducts is given in Fig. 14 (*b*). The sidewalk is carried on an outside bracket along the north truss; and the water main is located just inside the south truss, to help equalize the loads. The railway track is near the middle of the roadway, the trackway being covered with planking to provide a continuous surface. This planking is so detailed as to facilitate fitting the steel rails whenever the railway connections are completed.

In an earlier design, the trusses were spaced 31½ ft., center to center, with the water main located on an outside bracket. Careful studies showed that the small resulting increase in weight of steel was at the expense of other savings such as simplified shopwork and erection. The necessary revision increased the width to 33 ft. 6 in., center to center of trusses, or practically one-thirty-third of the span length.

On account of the inherent stability of the suspension construction, sway-bracing is unnecessary. Adequate systems of lateral bracing have been provided.

Design Loads

The dead load used in the design of the main span totaled 4 370 lb. per lin. ft., made up as follows:

Cables	750 lb.
Suspenders	20 "
Stringers	340 "
Floor-beams	300 "
Brackets	40 "
Trusses	1300 "
Bracing	230 "
Flooring	900 "
Railings	50 "
Rails	40 "
Water main	400 "
Total	4370 lb.

The live load was taken at 2 000 lb. per lin. ft., plus 10% for impact, or a total of 2 200 lb. per lin. ft., for the design of the stiffening trusses. For the design of the cables, which require full-span loading (and extreme temperature) for maximum stress, the live load was taken at 1 850 lb. per lin. ft.

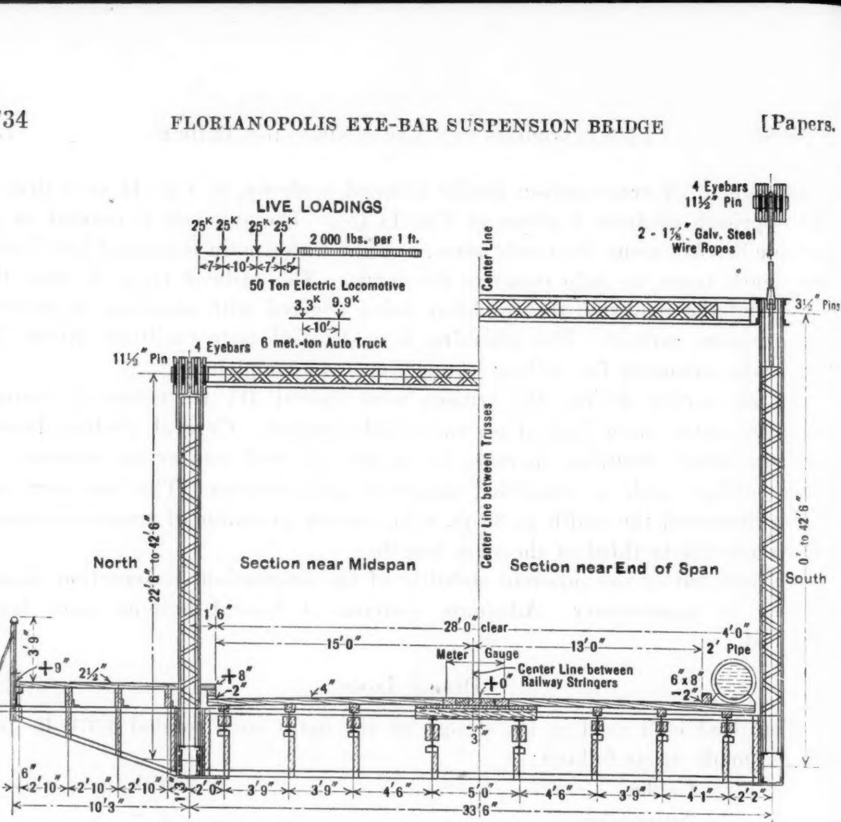
The floor was proportioned for the following moving loads:

Railway Loading.—A 50-ton electric locomotive followed by 2 000 lb. per lin. ft., plus 50% impact (Fig. 14).

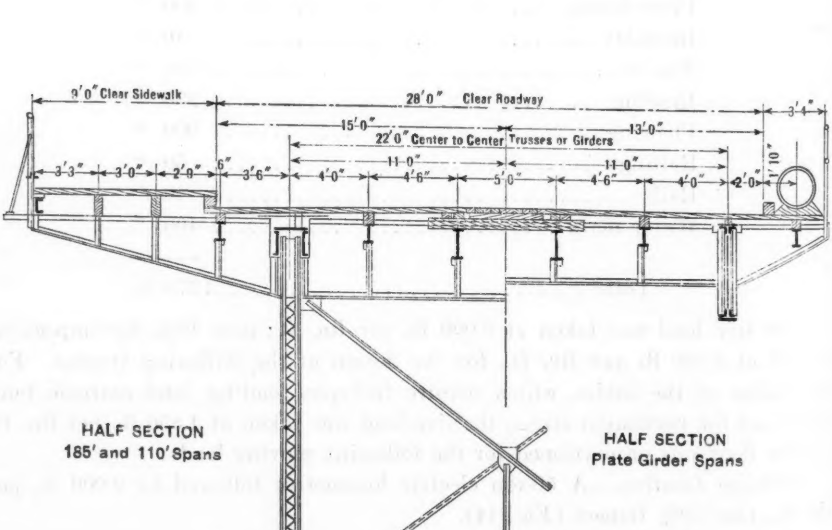
Highway Loading.—A 6-ton motor truck, or 60 lb. per sq. ft., with 25% impact (Fig. 14).

Sidewalk Loading.—60 lb. per sq. ft.

The wind load was taken at 25 lb. per sq. ft. on the main span, and 30 lb. per sq. ft. on the viaduct.



(*a*) Cross-Section of Main Span, Florianopolis Bridge.



(*b*) Cross-Section of Approach Viaducts.

Fig. 14.

The temperature variation was assumed as 30° Fahr., since the extreme temperatures reported for the locality are 30° and 90° Fahr., respectively.

Stresses by Method of Elastic Weights

The stresses in the main span of the Florianopolis Bridge were first calculated by the method of "elastic weights."

The first step is to calculate the stresses, u, produced in all the members of the unloaded structure by $H = 1$. For each truss member, that stress is given by $u = \frac{y}{r}$, in which, r equals the lever arm of the member about its center of moments; and y equals the vertical ordinate representing the respective effective lever arm of H.

The ordinate, y, is always measured along the vertical through the center of moments, and it is always measured from the closing chord of the cable. For the cable chord members, y is measured to the actual center of moments; for all other chord members, y is measured to the point of the chain directly above the actual center of moments; for the diagonals, y is measured from the closing chord of the cable to the prolongation of the cable member above the diagonal.

The strains, Δs, producible by the stresses, u, in the individual members are given by $\Delta s = \frac{u\,s}{E\,A}$, in which, s and A are the length and gross section of the member.

The next step is to calculate, for each truss member, the elastic weight, w, given by,

$$w = \frac{\Delta s}{r} = \frac{u\,s}{E\,A\,r} = \frac{y\,s}{E\,A\,r^2}$$

The calculations of the elastic weights, w, are recorded in Table 3. Each of these weights is considered as applied at the center of moments of the respective member, except that, in the case of a diagonal, the elastic weight, w, is resolved into two parallel opposing components, P and Q, applied at the respective ends of the diagonal:

$$P = -\,q\,\frac{w}{a};\text{ and } Q = p\,\frac{w}{a}$$

in which, p and q, differing by the panel length, a, are the respective distances of P and Q from w. These calculations are recorded in Table 4.

These elastic weights, w, for the chord members, and the component elastic weights, P and Q, for the diagonals, are combined and treated as applied simultaneously on the span. The resulting moment diagram or equilibrium polygon is the "elastic curve". It is the influence line for H, if all ordinates are divided by a constant N, given by $N = \Sigma\ (u\,\Delta s)$, in which, the summation extends over all members of the structure that are affected by H, including anchorage steel, towers, back-stays, cables, suspenders, and truss members. This calculation is recorded in Table 5.

TABLE 3.—Calculation of Elastic Weights for Chord Members.

Member.	Length, s, in feet.	Gross area, A, in square inches.	Modulus, E, in pounds per square inch.	Center of Moments.		Lever arm, r, in feet.	For H = Unity = 1 Lb.			
				Panel point.	Cable ordinate, y, in feet.		Stress, $u = \frac{y}{r}$, in pounds.	Stretch, $\Delta S = \frac{u\,s}{E\,A}$, in feet.	Elastic weight, $w = \Delta a = \frac{\Delta s}{r}$, in pounds.	$\Delta l = y\,\Delta a$, in foot-pounds.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
				Lower Chord.						
0-2	41.262	31.4	29 000 000	1	8.691	—11.247	—0.7727	—0.3502 × 10^{-7}	0.3113 × 10^{-8}	2.7057 × 10^{-8}
2-3	20.630	31.4	29 000 000	2	17.382	—22.495	—0.7727	—0.1751 "	0.0778 "	1.3527 "
3-5	41.260	41.4	29 000 000	4	33.471	—25.133	—1.3318	—0.4577 "	0.1821 "	6.0950 "
5-7	41.258	61.4	29 000 000	6	48.269	—27.839	—1.7339	—0.4018 "	0.1443 "	6.9658 "
7-9	41.256	61.4	29 000 000	8	61.716	—30.623	—2.0154	—0.4670 "	0.1525 "	9.4106 "
9-11	41.256	70.1	29 000 000	10	73.761	—33.480	—2.2031	—0.4471 "	0 1335 "	9.8504 "
11-13	41.254	70.1	29 000 000	12	84.358	—36.415	—2.3166	—0.4701 "	0.1291 "	10.8904 "
13-15	41.252	70.1	29 000 000	14	93.488	—39.428	—2.3711	—0.4812 "	0.1220 "	11.4086 "
15-17	41.252	70.1	29 000 000	16	101.147	—42.516	—2.3790	—0.4828 "	0.1136 "	11.4863 "
17-19	41.252	79.1	29 000 000	18	107.440	—35.841	—2.9977	—0.5391 "	0.1504 "	16.1601 "
19-21	41.250	86.6	29 000 000	20	112.472	—30.505	—3.6870	—0.6056 "	0.1985 "	22.3279 "
21-23	41.250	100.1	29 000 000	22	116.243	—26.505	—4.3857	—0.6232 "	0.2351 "	27.3322 "
23-25	41.250	100.1	29 000 000	24	118.751	—23.844	—4.9803	—0.7077 "	0.2968 "	35.2453 "
25-27	41.250	100.1	29 000 000	26	120.000	—22.519	—5.3288	—0.7572 "	0.3362 "	40.3500 "

TABLE 3.—(*Continued*).

Member.	Length, s, in feet.	Gross area, A, in square inches.	Modulus, E, in pounds per square inch.	CENTER OF MOMENTS.		Lever arm, r, in feet.	FOR H = UNITY = 1 LB.			
				Panel point.	Cable ordinate, y, in feet.		Stress, $u = \frac{y}{r}$, in pounds.	Stretch, $\Delta S = \frac{u\,s}{E\,A}$, in feet.	Elastic weight, $w = \Delta a = \frac{\Delta s}{r}$, in pounds.	$\Delta l = y\,\Delta a$, in foot-pounds.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
UPPER CHORD.										
0-1	23.735	74.7	29 000 000	1	8.691	+ 9.776	+0.8890	$+0.0974 \times 10^{-7}$	0.0996×10^{-8}	0.8659×10^{-8}
1-2	23.735	74.7	29 000 000	2	17.382	19.552	0.8890	0.0974 "	0.0498 "	0.8659 "
2-4	41.402	41.4	29 000 000	3	25.427	23.718	1.0720	0.3697 "	0.1559 "	3.9633 "
4-6	41.402	51.8	29 000 000	5	40.870	26.375	1.5496	0.4271 "	0.1619 "	6.6179 "
6-8	41.402	63.8	29 000 000	7	54.993	29.109	1.8892	0.4228 "	0.1452 "	7.9866 "
8-10	41.402	71.8	29 000 000	9	67.739	31.919	2.1222	0.4220 "	0.1322 "	8.9553 "
10-12	41.402	75.8	29 000 000	11	79.060	34.805	2.2715	0.4278 "	0.1229 "	9.7182 "
12-14	41.402	75.8	29 000 000	13	88.923	37.766	2.3546	0.4435 "	0.1174 "	10.4418 "
14-16	41.402	75 8	29 000 000	15	97.318	40.805	2.3850	0.4492 "	0.1101 "	10.7130 "
16-18 (CH)*	41.727	93.0	27 000 000	17	143.455	38.714	3.7055	0.6158 "	0.1591 "	22.8174 "
18-20 "	41.556	93.0	27 000 000	19	143.111	32.911	4.3484	0.7197 "	0.2187 "	31.2934 "
20-22 "	41.422	90.0	27 000 000	21	142.844	28.869	5.0352	0.8588 "	0.3026 "	43.2177 "
22-24 "	41.326	90.0	27 000 000	23	142.653	25.110	5.6811	0.9662 "	0.3848 "	54.8896 "
24-26 "	41.269	87.0	27 000 000	25	142.538	23.151	6.1569	1.0817 "	0.4672 "	66.5983 "
26-27 "	20.625	87.0	27 000 000	27	142.500	22.500	6.3333	0.5561 "	0.2472 "	35.2190 "
For chord members (½ truss), $\Sigma\,\Delta\,l$ = ...										525.7443 "

* $C\,H$ = Eye-bar chain.

TABLE 4.—CALCULATION OF ELASTIC WEIGHTS FOR DIAGONALS.

Diagonal.	Length, s, in feet.	Modulus, E, in pounds per square inch.	Gross area, A, in square inches.	CENTER OF MOMENTS.		Lever arm, r, in feet.	FOR H = 1 LB.	
				x, in feet.	y, in feet.		Stress, $u = \frac{y}{r}$, in pounds.	Stretch, $\Delta s = \frac{u \cdot s}{E A}$, in feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1- 2	23.260	29 000 000	14.7	0	0	+ 19.444	0	0
2- 3	30.179	29 000 000	14.7	− 314.346	−121.313	+ 280.500	−0.4325	−0.3062 × 10^{-7}
3- 4	32.858	29 000 000	14.7	− 306.178	−118.128	− 281.593	+0.4195	+0.3233 "
4- 5	32.179	29 000 000	14.7	− 306.178	−105.963	+ 319.749	−0.3314	−0.2501 "
5- 6	34.974	29 000 000	14.7	− 295.123	−101.997	− 317.068	+0.3217	+0.2639 "
6- 7	34.329	29 000 000	14.7	− 294.834	− 88.184	+ 356.235	−0.2475	−0.1993 "
7- 8	37.225	29 000 000	14.7	− 282.686	− 84.224	− 351.368	+0.2397	+0.2093 "
8- 9	36.624	29 000 000	14.7	− 282.996	− 69.099	+ 391.596	−0.1763	−0.1515 "
9-10	39.604	29 000 000	19.8	− 271.013	− 65.600	− 386.079	+0.1699	+0.1172 "
10-11	39.051	29 000 000	19.8	− 270.353	− 48.677	+ 426.339	−0.1142	−0.0776 "
11-12	42.104	29 000 000	19.8	− 257.978	− 45.498	− 419.389	+0.1085	+0.0796 "
12-13	41.605	29 000 000	19.8	− 257.648	− 27.448	+ 460.226	−0.0596	−0.0432 "
13-14	44.718	29 000 000	19.8	− 244.880	− 24.622	− 452.348	+0.0544	+0.0424 "
14-15	44.279	29 000 000	19.8	− 244.530	− 5.527	+ 493.247	−0.0112	−0.0086 "
15-16	47.445	29 000 000	19.8	− 231.062	− 3.027	− 484.316	+0.0063	+0.0052 "
16-17	47.068	29 000 000	19.8	+ 591.298	+141.010	− 217.408	−0.6486	−0.5317 "
17-18	41.503	29 000 000	19.8	+ 593.979	+141.419	+ 210.169	+0.6729	+0.4864 "
18-19	41.205	29 000 000	19.8	+ 646.264	+140.988	− 221.261	−0.6372	−0.4573 "
19-20	36.934	29 000 000	19.8	+ 649.914	+141.434	+ 213.118	+0.6636	+0.4269 "
20-21	36.713	29 000 000	14.7	+ 724.268	+140.973	− 241.911	−0.5828	−0.5019 "
21-22	33.660	29 000 000	19.8	+ 729.568	+141.458	+ 233.434	+0.6060	+0.3552 "
22-23	33.509	29 000 000	14.7	+ 858 990	+140.882	− 304.219	−0.4631	−0.3640 "
23-24	31.570	29 000 000	19.8	+ 869.839	+141.541	+ 298.691	+0.4789	+0.2605 "
24-25	31.484	29 000 000	14.7	+ 1 217.143	+140.617	− 531.279	−0.2647	−0.1955 "
25-26	30.551	29 000 000	19.8	+ 1 257.444	+141.837	+ 546.789	+0.2594	+0.1380 "
26-27	30.523	29 000 000	14.7	+24 981.206	+120.000	−18 019.568	−0.0067	−0.0048 "

TABLE 4.—(*Continued.*)

Elastic weight, $w = \frac{\Delta s}{r}$, in pounds.	P at Panel Point No.	Distance from c, p, in feet.	Elastic weight, $P = q \,.\, \frac{w}{a}$, in pounds.	Q at Panel Point No.	Distance from c, q, in feet.	Elastic weight, $Q = p \,.\, \frac{w}{a}$, in pounds.	$\Delta l = u \,\Delta s$, in foot-pounds.
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
0	1	20.625	0	2	41.250	0	0
−10.9152 × 10^{-11}	2	355.596	−0.1991 × 10^{-8}	3	376.221	0.1882 × 10^{-8}	1.3242 × 10^{-8}
−11.4825 "	3	368.053	−0.2164 "	4	388.678	0.2049 "	1.8564 "
− 7.8233 "	4	388.678	−0.1553 "	5	409.303	0.1474 "	0.8290 "
− 8.3237 "	5	398.248	−0.1690 "	6	418.873	0.1607 "	0.8490 "
− 5.5957 "	6	418.584	−0.1192 "	7	439.209	0.1136 "	0.4934 "
− 5.9570 "	7	427.061	−0.1293 "	8	447.686	0.1233 "	0.5017 "
− 8.8653 "	8	447.996	−0.0878 "	9	468.621	0.0840 "	0.2671 "
− 3.0354 "	9	456.638	−0.0702 "	10	477.263	0.0672 "	0.1991 "
− 1.8212 "	10	476.603	−0.0439 "	11	497.228	0.0421 "	0.0886 "
− 1.8968 "	11	484.853	−0.0465 "	12	505.478	0.0446 "	0.0863 "
− 0.9390 "	12	505.148	−0.0239 "	13	525.773	0.0230 "	0.0258 "
− 0.9871 "	13	513.005	−0.0242 "	14	533.630	0.0233 "	0.0231 "
− 0.1753 "	14	533.280	−0.0047 "	15	553.905	0.0045 "	0.0010 "
− 0.1066 "	15	540.437	−0.0029 "	16	561.062	0.0028 "	0.0003 "
+24.4548 "	16	261.282	−0.2853 "	17	240.656	0.3098 "	3.4484 "
+23.1412 "	17	243.354	−0.2499 "	18	222.729	0.2730 "	3.2726 "
+20.6661 "	18	275.014	−0.2549 "	19	254.389	0.2756 "	2.9137 "
+20.0298 "	19	258.039	−0.2306 "	20	237.414	0.2506 "	2.8329 "
+20.7459 "	20	311.768	−0.2928 "	21	291.143	0.3136 "	2.9246 "
+15.2176 "	21	296.443	−0.2035 "	22	275.818	0.2187 "	2.1527 "
+11.9655 "	22	405.240	−0.2231 "	23	384.615	0.2351 "	1.6857 "
+ 8.7227 "	23	395.464	−0.1585 "	24	374.839	0.1672 "	1.2346 "
+ 3.6793 "	24	722.143	−0.1251 "	25	701.518	0.1288 "	0.5174 "
+ 2.5241 "	25	741.819	−0.0883 "	26	721.194	0.0908 "	0.3580 "
+ 0.0026 "	26	24 444.956	−0.0031 "	27	24 424.331	0.0031 "	0.0003 "
For diagonals (½ truss), $\Sigma\, \Delta\, l$ =							27.3859 "

TABLE 5.—CALCULATION OF Δl FOR ANCHORAGE STEEL, CABLE, SUSPENDERS, BACK-STAYS, AND TOWERS.

Member.		Length, s, in feet.	Modulus, E, in pounds per square inch.	Gross area A, in square inches.	For $H = 1$ Lb.			
					Stress, u, in pounds.	Stretch, $\Delta s = \frac{u\,s}{E\,A}$, in feet.	$\Delta l = u \Delta s$ in foot-pounds, for one-half truss.	Δl, for one truss.
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)
Cable	0-2	44.763	27 000 000	96	1.0852	1.8740×10^{-8}	2.0336×10^{-8}	....
	2-4	44.277	27 000 000	96	1.0734	1.8336 "	1.9681	....
	4-6	43.824	27 000 000	93	1.0624	1.8542 "	1.9699	....
	6-8	43.386	27 000 000	93	1.0518	1.8173 "	1.9114	....
	8-10	42.978	27 000 000	93	1.0418	1.7829 "	1.8573	....
	10-12	42.589	27 000 000	93	1.0325	1.7512 "	1.8080	....
	12-14	42.248	27 000 000	90	1.0242	1.7807 "	1.8237	....
	14-16	41.955	27 000 000	90	1.0171	1.7560 "	1.7861	....
$\Sigma \Delta l =$							15.1581	30.3162×10^{-8}
Suspenders	2	108.584	20 000 000	3.4706	0.0313	4.9035×10^{-8}	0.1537	....
	4	88.949	20 000 000	3.4706	0.0313	4.0106 "	0.1255	....
	6	70.606	20 000 000	3.4706	0.0328	3.3315 "	0.1091	....
	8	53.613	20 000 000	3.4706	0.0340	2.6252 "	0.0892	....
	10	38.023	20 000 000	3.4706	0.0351	1.9229 "	0.0675	....
	12	23.880	20 000 000	3.4706	0.0356	1.2285 "	0.0435	....
	14	11.205	20 000 000	3.4706	0.0357	0.5757 "	0.0205	....
$\Sigma \Delta l =$							0.6090	1.2180×10^{-8}

TABLE 5.—(*Continued*).

Member.		Length, s, in feet.	Modulus E, in pounds per square inch.	Gross area, A, in square inches.	For H = 1 Lb.			
					Stress, u, in pounds.	Stretch, $\Delta s = \frac{u\,s}{A\,E}$, in feet.	$\Delta l = u \Delta s$ in foot-pounds, for one-half truss.	Δl, for one truss.
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)
Mainland tower	El. 20.5–El. 88.0	67.584	29 000 000	342.8	−0.5764	-0.3919×10^{-8}	....	0.2259×10^{-8}
	El. 88.0–El. 241.0	153.190	29 000 000	346.0	−0.9983	−1.5241 "	....	1.5215 "
Island tower	El. 20.5–El. 88.0	67.584	29 000 000	342.8	−0.5614	−0.3816 "	....	0.2143 "
	El. 88.0–El. 241.0	153.190	29 000 000	346.0	−0.9833	−1.5012 "	....	1.4761 "
Mainland back-stay		490.144	27 000 000	96.0	....	....	....	24.5734 "
Island back-stay		425.930	27 000 000	96.0	....	....	....	21.1445 "
Mainland anchorage steel		....	27 000 000	......	....	....	....	0.8808 "
Island anchorage steel		....	27 000 000	......	....	....	....	0.8758 "
For diagonals, $\Sigma \Delta l$ =								54.7718×10^{-8}
For chords, $\Sigma \Delta l$ =								$1\,051.4886 \times 10^{-8}$
Total, δ =								$1\,188.7069 \times 10^{-8}$

It is to be noted that the foregoing procedure is equivalent to constructing the H-curve to satisfy the influence-line equation,

$$H = \frac{\Sigma\,(Z\,\Delta\,s)}{\Sigma\,(u\,\Delta\,s)}$$

in which, Z equals the stress producible in each truss member by a unit concentrated load traversing the span, if the span were a simple truss without a cable. The elastic weight, w, for any member can be defined as that imaginary concentrated weight which must be placed on the span to yield a simple-beam moment diagram identical with the influence line for $Z\,\Delta\,s$ for that member. The equilibrium polygon for all the elastic weights, w, combined is the summation of the individual influence lines for $Z\,\Delta\,s$ and is, therefore, the influence line for $\Sigma\,(Z\,\Delta\,s)$, the numerator of the H-equation.

On the elastic curve, or H-curve, constructed as described (see Table 6), the simple-span straight-line influence diagrams for the various truss members (drawn to corresponding scale) are superimposed, and the intercepted areas are the required influence areas for the respective members.

TABLE 6.—CALCULATION OF ORDINATES TO H-CURVE.

Panel point.	Elastic weight for chord members, L-lower, U-upper, in pounds.	ELASTIC WEIGHTS FOR DIAGONALS.		Total elastic weight, in pounds.	Shear, in pounds.	Moment, in foot-pounds.	H, ordinates.
		P, in pounds.	Q, in pounds.				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	L 0.3113 × 10^{-8} U 0.0996 "	....	....	0.4109 × 10^{-8}	5.5462 × 10^{-8}	114.39 × 10^{-8}	0.0962
2	L 0.0778 " U 0.0498 "	−0.1991 × 10^{-8}	....	−0.0715 "	5.1353 "	220.31 "	0.1853
3	U 0.1559 "	−0.2164 "	0.1882 × 10^{-8}	0.1277 "	5.2068 "	327.70 "	0.2757
4	L 0.1821 "	−0.1553 "	0.2049 "	0.2317 "	5.0791 "	432.45 "	0.3638
5	U 0.1619 "	−0.1690 "	0.1474 "	0.1403 "	4.8474 "	532.43 "	0.4479
6	L 0.1443 "	−0.1192 "	0.1607 "	0.1858 "	4.7071 "	629.51 "	0.5296
7	U 0.1452 "	−0.1293 "	0.1136 "	0.1295 "	4.5213 "	722.77 "	0.6080
8	L 0.1525 "	−0.0878 "	0.1233 "	0.1880 "	4.3918 "	813.35 "	0.6842
9	U 0.1322 "	−0.0702 "	0.0840 "	0.1460 "	4.2038 "	900.05 "	0.7572
10	L 0.1335 "	−0.0439 "	0.0672 "	0.1568 "	4.0578 "	983.74 "	0.8276
11	U 0.1229 "	−0.0465 "	0.0421 "	0.1185 "	3.9010 "	1064.20 "	0.8953
12	L 0.1291 "	−0.0239 "	0.0446 "	0.1498 "	3.7825 "	1142.21 "	0.9609
13	U 0.1174 "	−0.0242 "	0.0230 "	0.1162 "	3.6327 "	1217.14 "	1.0239
14	L 0.1220 "	−0.0047 "	0.0233 "	0.1406 "	3.5165 "	1289.67 "	1.0849
15	U 0.1101 "	−0.0029 "	0.0045 "	0.1117 "	3.3759 "	1359.29 "	1.1435
16	L 0.1136 "	−0.2853 "	0.0028 "	−0.1689 "	3.2642 "	1426.62 "	1.2001
17	U 0.1591 "	−0.2499 "	0.3098 "	0.2190 "	3.4331 "	1497.43 "	1.2597
18	L 0.1504 "	−0.2549 "	0.2730 "	0.1685 "	3.2141 "	1563.72 "	1.3155
19	U 0.2187 "	−0.2306 "	0.2756 "	0.2637 "	3.0456 "	1626.53 "	1.3683
20	L 0.1985 "	−0.2928 "	0.2506 "	0.1563 "	2.7819 "	1683.91 "	1.4166
21	U 0.3026 "	−0.2035 "	0.3136 "	0.4127 "	2.6256 "	1738.06 "	1.4621
22	L 0.2351 "	−0.2231 "	0.2187 "	0.2307 "	2.2129 "	1783.70 "	1.5005
23	U 0.3848 "	−0.1585 "	0.2351 "	0.4614 "	1.9822 "	1824.59 "	1.5349
24	L 0.2968 "	−0.1251 "	0.1672 "	0.3389 "	1.5208 "	1855.95 "	1.5613
25	U 0.4672 "	−0.0883 "	0.1288 "	0.5077 "	1.1819 "	1880.33 "	1.5818
26	L 0.3362 "	−0.[illegible]31 "	0.0908 "	0.4239 "	0.6742 "	1894.24 "	1.5935
27	U 0.2472 "	....	0.0031 "	0.2503 "	0.2503 "	1899.40 "	1.5979

The influence diagrams obtained in this manner for the chord members are reproduced in Fig. 15. Similar influence diagrams for the diagonals were drawn, but are not reproduced here. The influence diagrams for the upper chord members in the middle part of the span clearly show the stresses reduced as a result of the cable combination.

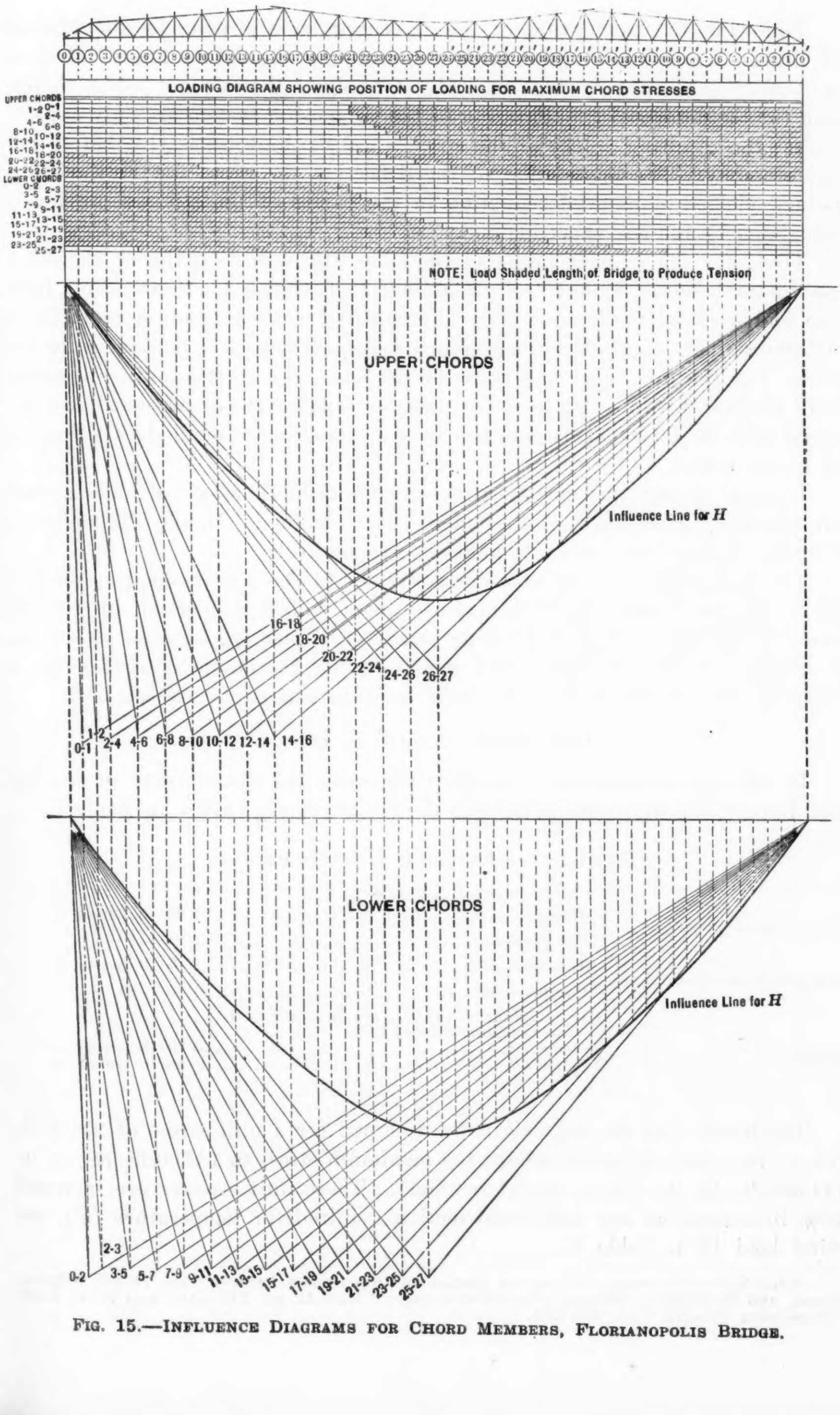


FIG. 15.—INFLUENCE DIAGRAMS FOR CHORD MEMBERS, FLORIANOPOLIS BRIDGE.

Stresses by the Deflection Theory

Following the calculation of the stiffening truss stresses by the Method of Elastic Weights, a re-calculation was made by the "More Exact" Method, or Deflection Theory.* The methods of analysis that do not make correction for the deformed configuration of the suspension system are only approximate; the resulting values of the stresses are too high, satisfying safety but not economy. The application of the deflection theory to the Florianopolis Bridge yielded a material reduction in the values of the stresses previously calculated by the less exact method.

Thus, for the quarter-points of the span, the deflection theory showed a maximum bending moment of 24 367 000 ft-lb. (with load extending from one end over 0.450 of the span), as compared with a bending moment of 30 300 000 ft-lb. obtained by the approximate method—a reduction of 20 per cent. For the middle or half-point of the span, the corresponding moments were 16 929 000 ft-lb. (with load covering the middle 0.460 of the span), as compared with 26 800 000 ft-lb. obtained by the approximate method—a reduction of 37 per cent.

In more flexible suspension spans, greater reductions of calculated stress are found by application of the "more exact" method. In the Philadelphia-Camden Bridge, these were 34 and 38%, respectively.

The final values of the maximum stresses in the Florianopolis stiffening truss, by the "more exact" theory, are about 14 500 lb. per sq. in. for the assumed loading. Since a working stress of 18 500 lb., or even 20 000 lb., is amply safe for stiffening truss members, the Florianopolis Bridge has a capacity for concentrated loads considerably in excess of that specified.

Unit Stresses Used in Design

In consideration of the comparatively generous assumptions of loading and impact, the unit stresses used in the design were as given in Table 7.

TABLE 7.—Assumed Unit Stresses.

Floor system	Tension	17 000 lb. per sq. in.
	Compression	$17\,000 - 80 \frac{l}{r}$
Stiffening truss, towers, etc.	Tension	18 500 lb. per sq. in.
	Compression	$18\,500 - 85 \frac{l}{r}$
	For combination of $D + L + I$ and $T + W$, increase unit stresses 25%.	
Suspenders	Tension	55 000 lb. per sq. in.
Chains	Tension	46 500 lb. per sq. in.

The towers and trusses, originally designed for a unit stress of 20 000 lb. per sq. in., were revised to reduce the maximum stress to 18 500 lb. per sq. in. (14 500 lb. by the "more exact" method). These unit stresses are increased 25% in computing any additional section required for temperature (T), and wind load (W), Table 8.

* See Melan-Steinman, "Theory of Arches and Suspension Bridges," pp. 76–86; Johnson, Bryan, and Turneaure, "Modern Framed Structures," Part II, pp. 276–318; and W. H. Burr; "Suspension Bridges," pp. 212–247.

TABLE 8.—DESIGN OF STIFFENING TRUSSES.

CHORDS.															DIAGONALS.						
Maximum stress.								Section.							Maximum stress.				Section.		
								Size, in inches.				Area, in square inches.									
Member.	D.	L + I.		T.	W.		Total.		Two web-plates.	Four outside angles.	Two inside plates.	Two outside plates.	Gross.	Net.	Member.	D + L + I.		T.	Two channels.	Gross area, in feet.	Net area, in feet.
U0- 2	0	+ 224	−414	± 27	......	......	+ 251	− 441	24 by ¾	4 by 4 by ¾	(Cover, 27 by ⅝)		74.7	61.2	D 1-2	......	− 95	0	......	....	....
2- 4	0	+ 260	−492	± 32	......	− 81	+ 292	− 605	24 by ⅝	4 by 4 by ⅜	......	......	41.4	34.0	2-3	+198	−100	±13	......	....	....
4- 6	0	+ 348	−678	± 46	+ 81	−115	+ 470	− 839	24 by ⅝	4 by 4 by ¾	......	......	51.8	42.5	3-4	+102	−172	±12	......	....	....
6- 8	0	+ 372	−776	± 57	+155	−223	+ 584	−1 056	24 by ⅝	4 by 4 by ¾	......	16 by ⅜	63.8	52.3	4-5	+153	− 72	±10	......	....	....
8-10	0	+ 365	−823	± 63	+223	−284	+ 651	−1 170	24 by ⅝	4 by 4 by ¾	......	16 by ⅜	71.8	59.7	5-6	+ 82	−131	±10	12-in. by 25-lb.	14.7	12.1
10-12	0	+ 341	−828	± 67	+284	−338	+ 692	−1 233	24 by ⅝	4 by 4 by ¾	......	16 by ¾	75.8	62.2	6-7	+126	− 61	± 7	......	....	....
12-14	0	+ 297	−796	± 70	+338	−385	+ 705	−1 251	24 by ⅝	4 by 4 by ¾	......	16 by ¾	75.8	62.2	7-8	+ 76	−109	± 7	......	....	....
14-16	0	+ 244	−755	± 71	+385	−425	+ 700	−1 251	24 by ⅝	4 by 4 by ¾	......	16 by ¾	75.8	62.2	8-9	+114	− 67	± 5	......	....	....
16-18	+2 863	+ 771	−281	±112	+425	−460	+4 171	......	......	......	......	......	93.0	93.0	9-10	+ 86	−103	± 5	......	....	....
18-20	+2 849	+ 686	−322	±133	+460	−486	+4 128	......	......	......	......	......	93.0	93.0	10-11	+120	− 86	± 3	......	....	....
20-22	+2 837	+ 577	−356	±155	+486	−507	+4 055	......	......	......	......	......	90.0	90.0	11-12	+103	−111	± 3	......	....	....
22-24	+2 832	+ 455	−375	±176	+507	−520	+3 970	......	......	......	......	......	90.0	90.0	12-13	+131	−105	± 2	......	....	....
24-26	+2 830	+ 356	−357	±191	+520	−527	+3 897	......	......	......	......	......	87.0	87.0	13-14	+123	−121	± 2	......	....	....
26-26′	+2 823	+ 318	−332	±196	+527	−527	+3 864	......	......	......	......	......	87.0	87.0	14-15	+140	−123	± 1	15-in. by 33-lb.	19.8	16.2
										Four inside angles.	......	......	......	....	15-16	+198	−186	± 1	......	....	....
L 0- 2	0	+ 378	−210	± 23	0	− 88	+ 401	− 321	20 by ½	4 by 4 by ⅜	......	......	31.4	25.9	16-17	+195	− 85	±21	......	....	....
2- 3	0	+ 362	−197	± 23	+ 88	−169	+ 473	− 389	20 by ½	4 by 4 by ⅜	......	......	31.4	25.9	17-18	+ 41	−177	±22	......	....	....
3- 5	0	+ 592	−308	± 40	+169	−244	+ 801	− 592	20 by ¾	4 by 4 by ⅜	......	......	41.4	33.9	18-19	+207	− 39	±20	......	....	....
5- 7	0	+ 730	−364	± 52	+244	−311	+1 026	− 727	20 by ¾	4 by 4 by ⅜	......	20 by ½	61.4	49.9	19-20	+ 51	−193	±21	......	....	....
7- 9	0	+ 800	−375	± 60	+311	−372	+1 171	− 807	20 by ¾	4 by 4 by ⅜	......	20 by ½	61.4	49.9	20-21	+220	− 69	±19	12-in. by 25-lb.	14.7	12.1
9-11	0	+ 818	−360	± 65	+372	−425	+1 255	− 850	20 by ¾	4 by 4 by 11/16	......	20 by ½	70.1	57.4	21-22	+ 82	−210	±20	15-in. by 33-lb.	19.8	16.2
11-13	0	+ 811	−325	± 68	+425	−474	+1 304	− 867	20 by ¾	4 by 4 by 11/16	......	20 by ½	70.1	57.4	22-23	+231	−111	±15	12-in. by 25-lb.	14.7	12.1
13-15	0	+ 773	−275	± 70	+474	−514	+1 317	− 859	20 by ¾	4 by 4 by 11/16	......	20 by ½	70.1	57.4	23-24	+128	−220	±15	15-in. by 33-lb.	19.8	16.2
15-17	0	+ 717	−220	± 71	+514	−548	+1 302	− 839	20 by ¾	4 by 4 by 11/16	......	20 by ½	70.1	57.4	24-25	+235	−155	± 8	12-in. by 25-lb.	14.7	12.1
17-19	0	+ 846	−207	± 91	+548	−575	+1 485	− 873	20 by ¾	4 by 4 by 11/16	12 by ⅜	20 by ½	79.1	64.9	25-26	+172	−221	± 8	15-in. by 33-lb.	19.8	16.2
19-21	0	+ 962	−174	±110	+575	−595	+1 647	− 879	20 by ¾	4 by 4 by 11/16	12 by 11/16	20 by ½	86.6	71.2	26-27	+218	−192	0	12-in. by 25 lb.	14.7	12.1
21-23	0	+1 058	−120	±136	+595	−608	+1 789	− 864	20 by ¾	4 by 4 by 11/16	12 by 11/16 18 by ⅜	20 by ½	100.1	81.7	......	......	......	....	......	....	....
23-25	0	+1 140	− 73	±154	+608	−615	+1 902	− 842	20 by ¾	4 by 4 by 11/16	12 by 11/16 18 by ⅜	20 by ½	100.1	81.7	......	......	......	....	......	....	....
25-27	0	+1 180	− 33	±165	+615	−615	+1 960	− 813	20 by ¾	4 by 4 by 11/16	12 by 11/16 18 by ⅜	20 by ½	100.1	81.7	......	......	......	....	......	....	....

Maximum stress in suspenders, in 1 000 lb. $\left\{\begin{array}{l} D = 85 \\ L = 74 \\ I = 31 \end{array}\right\}$, total, 190.

Maximum stress in verticals, in 1 000 lb.......... $\left\{\begin{array}{l} D = +30 \\ L = +44 \\ I = +19 \end{array}\right\}$, total + 93.

Sections of Truss Members

The sections used for the members of the stiffening trusses are shown in Table 8.

The top chord sections are composed of two built-up channels, $24\frac{1}{2}$ in. high and $17\frac{1}{4}$ in. apart, with flanges turned out. The bottom chord sections are composed of two built-up channels, $20\frac{1}{2}$ in. high by $16\frac{1}{4}$ in. wide, with flanges turned in. The diagonals are made up of two 12 or 15-in. channels, with flanges turned in. All verticals (functioning only as hangers) are four-angled sections, with single plane of lacing, providing a net section of 8.8 sq. in. to carry a maximum tension of 93 000 lb.

The details of one of the top chord members (*U*-14-*U*-16 which connects to the chain at *U*-16) are shown in Fig. 17.

Sections of Eye-Bars

Each eye-bar cable consisted of four similar members of 12-in. depth and of widths varying from 2 to $1\frac{13}{16}$ in. The methods of compilation have already been described. For convenience the details of stresses and sections have been listed in Table 9.

TABLE 9.—Design of Chains.

Member.	Maximum stress, in thousands of pounds.	Four eye-bars, size, in inches.	Section, in square inches.
Back-stays	+4 381	12 by 2	96
0–2	+4 179	12 by 2	96
2–4	+4 133	12 by 2	96
4–6	+4 094	12 by $1\frac{15}{16}$	93
6–8	+4 059	12 by $1\frac{15}{16}$	93
8–10	+4 024	12 by $1\frac{15}{16}$	93
10-12	+3 990	12 by $1\frac{15}{16}$	93
12–14	+3 959	12 by $1\frac{7}{8}$	90
14-16	+3 932	12 by $1\frac{7}{8}$	90
16–18	+4 171	12 by $1\frac{15}{16}$	93
18–20	+4 128	12 by $1\frac{15}{16}$	93
20–22	+4 055	12 by $1\frac{7}{8}$	90
22–24	+3 970	12 by $1\frac{7}{8}$	90
24–26	+3 897	12 by $1\frac{13}{16}$	87
26–26′	+3 864	12 by $1\frac{13}{16}$	87

The Approach Viaducts

The cross-section of the two viaduct approaches is shown in Fig. 14 (*b*). Immediately flanking the main suspension span are 185-ft. deck trusses, spanning from each main tower to the near-by shore. These deck spans were used for tying in and holding the main rocker towers during erection. Symmetry terminates with the 30-ft. tower spans flanking the 185-ft. deck spans. Each approach also contains a 110-ft. deck-truss span, over existing highways.

The remainder of each viaduct is made up, as far as practicable, of 60-ft. girder spans and 30-ft. tower spans; each anchorage serves to support one of the latter. An odd 45-ft. girder span also occurs in each viaduct,

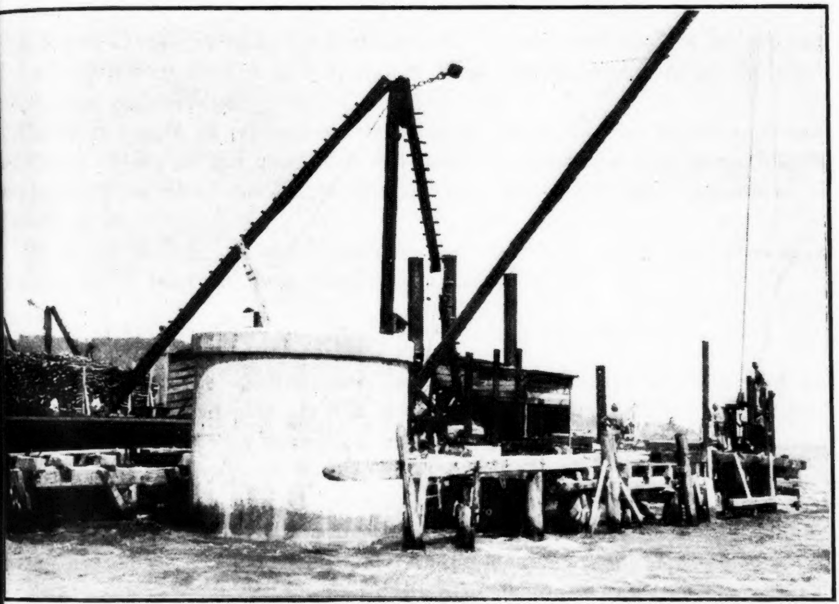


FIG. 16.—VIEW OF ONE OF THE MAIN PIERS, COMPLETED, FLORIANOPOLIS BRIDGE.

FIG. 17.—TOP CHORD DETAIL AT U-16, FLORIANOPOLIS BRIDGE.

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adjoining the anchorage on the island side and adjoining a bent on the continent side.

A length of 135 ft. near the east end of the continent viaduct is laid out to a curved center line of 150-m. radius (Fig. 5), for easement of the electric railway connections.

The total length of viaduct construction is 850 ft. on the island approach and 726 ft. 6 in. on the continent approach. Adding the main-span length of 1 113 ft. 9 in., the total length of structure from abutment to abutment is 2 690 ft. 3 in.

The grade is $2\frac{1}{2}\%$ on each approach, the main span being cambered to a parabolic curve between these tangent gradients.

Development of Design of Viaducts and Towers

The design of the viaducts was normal for that class of work and no special problems developed in the preparation of the detail plans except where the tower columns were tied in to the adjoining 185-ft. spans during the erection of the towers.

On account of the hinged bases the main columns had to be supported during erection until such time as they were held definitely in position by the eye-bar cables. On account of shipping and erection requirements the weight of each column section was limited to about 15 gross tons, or 34 000 lb. There were, therefore, ten column sections ranging in length from 20 to 25 ft., with an average length of 22 ft. for each of the four main columns, the elevation of the top of the fourth section being somewhat higher than the viaduct floor level. Provision was made to connect the top of the second column section back to the end bottom chord, Point *L*-1, of the 185-ft. span, with a temporary strut until the third and fourth column sections were placed; then to connect the top of the fourth column section back to the corresponding *U*-1 point with a second temporary strut and to remove the first. Several of the permanent bottom laterals of the suspension span were used for these temporary struts. The second or upper temporary struts were to be removed after the eye-bar chains were swung.

Rivets of 1 in. diameter for both shop and field work were used in the main columns. To insure the tower members fitting together easily in the field each tower was assembled complete in the yards of the Elmira Plant and the field connections reamed and match-marked.

Development of Design of Eye-Bars and Pins

The suspension span was designed so that the entire dead load was carried by the eye-bar cables. This dead load was not uniform either along the cable or along the horizontal so that the pins connecting the eye-bars lay neither in a catenary nor in a parabola. The position of the eye-bar cable in space was an equilibrium polygon passing through three fixed points, the top of each tower and a point at the center of the span having a sag of 120 ft.

Given the distance of 1 113 ft. 9 in., center to center of towers, a sag of 120 ft., twenty-seven equi-distant panels of 41 ft. 3 in. each, and twenty-six panel loads, of amounts varying from 85 000 to 98 400 lb., but symmetrical about the center line of span, it was possible to calculate the ordinates of the equilibrium polygon and the lengths of its rays. These lengths represented the lengths of the eye-bars under full dead load stress. The horizontal component of the eye-bar cable stress, which was constant from anchorage to anchorage, was calculated by the usual method and from this the stresses in the other eye-bar panels were computed. The elongation of each panel of eye-bars was computed from the formula, $\lambda = \frac{P\,l}{a\,E}$, using a value of 27 000 000 for E. The lengths of the eye-bars under dead load were shortened by the amounts of these elongations and the eye-bars were fabricated to these shortened lengths, center to center of pin-holes, making due allowance for the small amount of pin play. Table 10 gives the panel loads.

TABLE 10.—DEAD LOAD PANEL CONCENTRATIONS FOR EYE-BAR CABLE, IN POUNDS.

Item.	Panel Point.												
	2	4	6	8	10	12	14	16	18	20	22	24	26
Cable	17 790	17 400	16 950	16 890	16 700	16 390	15 980	16 170	16 390	16 060	15 710	15 450	15 240
Suspenders	2 220	2 000	1 780	1 570	1 400	1 220	1 060	......	......	......	......	......	......
Truss	22 360	22 460	27 130	29 750	33 160	34 290	34 890	30 820	23 030	23 040	24 360	24 350	23 990
Bracing	6 030	4 840	4 640	4 390	4 240	4 000	3 770	3 710	3 680	3 600	3 530	3 500	3 470
Floor steel	15 500	15 500	15 500	15 500	15 500	15 500	15 500	15 500	15 500	15 500	15 500	15 500	15 500
Floor timber	16 400	16 400	16 400	16 400	16 400	16 400	16 400	16 400	16 400	16 400	16 400	16 400	16 400
Railing	1 000	1 000	1 000	1 000	1 000	1 000	1 000	1 000	1 000	1 000	1 000	1 000	1 000
Water main	9 000	9 000	9 000	9 000	9 000	9 000	9 000	9 000	9 000	9 000	9 000	9 000	9 000
Rails	800	800	800	800	800	800	800	800	800	800	800	800	800
Total load per cable	91 100	89 400	93 200	95 300	98 200	98 600	98 400	93 400	85 800	85 400	86 300	86 000	85 400

For connecting the 12-in. eye-bars of the cables, pins, $11\frac{1}{2}$ in. in diameter, were used; consequently, with eye-bar heads the same thickness as the body of the bar, the bearing pressure on these pins would have exceeded the working tension in the eye-bars. To reduce this high unit bearing pressure the heads were made $\frac{1}{8}$ in. thicker than the remainder of the bars, thus reducing the unit pressure from 6 to 9%, depending on the bar thickness. To resist the high unit stress the pins were made of special heat-treated steel with a yield point ranging between 60 000 and 65 000 lb. per sq. in., and a tensile strength ranging between 100 000 and 105 000 lb. per sq. in. A few of the pins are of chrome-nickel steel having the same range of strength.

To facilitate the entry of the pins during erection, a novel detail in the form of oval pin-holes was developed. (Fig. 18.) The hole is made somewhat elongated axially and enlarged on the inside, to provide more clearance for the insertion of the pin or for slipping the bar over the pin, while

retaining a close fit in the segment in contact. The outer or bearing semicircle exceeds the diameter of the pin by only 0.005 in., while the inner semicircle is bored to the diameter of the pin plus $\frac{1}{32}$ in., and the two centers are separated $\frac{1}{8}$ in. along the axis of the eye-bar. The unusually close fit thus secured along the bearing surface reduces the secondary stresses in the eye-bar head. This detail is patented* by the American Bridge Company.

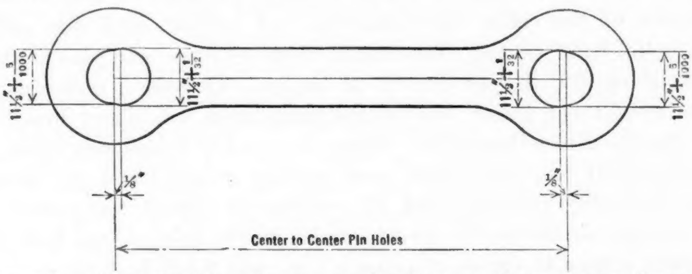


FIG. 18.—NOVEL PIN-HOLE DETAIL FOR EYE-BARS OF FLORIANOPOLIS BRIDGE.

DEVELOPMENT OF DESIGN OF SUSPENSION SPAN TRUSSES

The decision to make the central part of the top chord integral with the eye-bar cable presented some interesting studies in fabrication and erection. According to the design the eye-bar cable was to support the entire dead weight of the bridge; and the stiffening truss was designed for live load, wind, and temperature. This meant that when the entire dead load was suspended from the eye-bar cables they would occupy a definite fixed position in space at a given normal temperature and with no wind blowing. Top chord Points 2 to 14, inclusive, at each end of the truss, are supported from the cables by rope hangers, while top chord Points 16 to 16 are parts of the eye-bar cables.

The rope hangers are made with provision for permanent adjustment whereas between Points 16 and 16 there are no means of adjustment. For this reason all diagonal members of the truss between Points 16 were shipped with rivet holes in both ends of the diagonals; but the gusset-plates of the bottom chords to which these diagonals connected, were shipped blank. After the entire dead load was supported from the eye-bar cables the rivet holes could be drilled in these gusset-plates using the holes in the diagonals as templets and thus insuring the erection of the truss members under zero dead load stress.

This meant the drilling of 1 728 field holes; but as the material was in every case the ½-in. gusset-plates, it was considered safer to do the field drilling than to run the risk of poorly matched holes which would involve either drifting and the introduction of dead load stress in the truss members or the necessity of excessive reaming.

The truss and bracing members were of normal sizes for heavy highway work and presented no special difficulties in the shop. In fact, the entire structure was fabricated without unusual difficulties.

* Patent No. 1,600,233, filed July 2, 1924, Serial No. 723,761.

Development of Method for Erecting Cables

The success of the construction was dependent on the successful erection of the eye-bars forming the main cables. When the entire dead load was carried by the cables the sag at the center should be 120 ft. with the main towers vertical. At that time the dead load of the main span, including the cables, was about 4 400 lb. per lin. ft. of bridge, and the dead load of the back-stay portion of the cable, which consisted of eye-bars and pins only, was 900 lb. per lin. ft. of bridge. The weight of the eye-bars and pins of the main span was about 850 lb. per lin. ft. of bridge. Therefore, under dead load the total weight per linear foot of the main span was about five times as much as for the cables themselves. When the eye-bar cables were swung under their own weight only, the main span portions weighed less per linear foot than the back-stay portions, and the calculated sag at the center of the main span was about 116 ft., based on the towers being tilted back so that the distance, center to center of column tops, was 1 115 ft. 7 in.

To make the erection of the eye-bars as simple and cheap as possible it was decided to hang them by hand-operated chain hoists which in turn were suspended from flexible ropes; then, after all the eye-bars and pins were placed and supported by the ropes, to swing the eye-bar cables by paying out the chain hoists. To make this vertical release movement of the eye-bars as small as possible, the ropes were made of such length that when all the bars were supported, the sag of the ropes (and, hence, of the bars) should be 115 ft., just 1 ft. less than the sag of the bars swinging under their own weight. The ropes had to support themselves as well as the bars. The weight on the main span portion of the ropes, consisting of the eye-bars, pins, ropes, chain hoists, clamps, and connections, was about 500 lb. per lin. ft of each cable.

To save as much erection material as possible flexible hoisting ropes, 1 in. in diameter, were used, so that after having served their purpose in the erection of the two eye-bar cables they could be cut up and used for ordinary hoisting rope in the Erecting Department. The grade selected had an approximate ultimate strength of 90 000 lb. per rope; a factor of safety of three was used for erection. Based on a sag of 115 ft. and a span of 1 115 ft. 7 in., the horizontal component of the stress in the ropes when they supported all the eye-bars was 660 000 lb. and the maximum stress that occurred in the anchorage portion of the ropes was 770 000 lb. At 32 000 lb. per rope, twenty-four 1-in. ropes were required.

After their completion the main towers will rock on their pedestals (see Fig. 10) under various conditions of loading in such manner as to make the horizontal components of the cable stresses constant from anchorage to anchorage and so give resultant vertical loads only. Therefore, the saddles (Fig. 7) were fixed to the tops of the columns. During the erection of the eye-bars, however, the main tower columns were locked into the 185-ft. spans. To insure that the horizontal components of the stresses in the erection rope would be taken entirely by the anchorages, temporary I-beam grillages were provided, bearing on rollers which in turn rested on top of the column castings (Fig. 29).

Wire rope is much more elastic than ordinary structural steel. It was necessary to ascertain either the modulus of elasticity of the ropes or their elongation under a stress of 32 000 lb. per rope. The lengths of the erection ropes, under full load, had then to be shortened by the amount of this elongation so that when they supported all the eye-bars, the sag at the center would not exceed 115 ft. Ordinarily, the rope used for suspension bridge cables consisting of 6 strands with a wire center, has a modulus of elasticity of from 12 000 000 to 20 000 000 depending upon the lay of the rope and the stress it carries. Ordinary flexible hoisting rope consisting of 6 strands with a hemp center has a modulus of elasticity of from 5 000 000 to 10 000 000.

The method adopted for the erection of the eye-bars was to attach them and their pins to the lower ends of the hand-operated chain hoists, the upper ends of which were held by clamps around the ropes. The load at each panel point was 19 000 lb., so that hoists with capacities of 20 000 lb. were selected. When all bars were suspended, the chain hoists could be played out and the eye-bars lowered from a sag of 115 ft. until they were carrying their own weight under a sag of 116 ft. As the loads on the hoists were released, the erection ropes were relieved of their stress, and shortened to a length corresponding to their own weight but not to their original length because of the permanent set they had acquired. The sag meanwhile decreased from 115 ft. until it reached an amount dependent on the new length of the ropes. This decrease had to be determined in order to provide the proper amount of chain for the operation of the chain hoists. The greatest vertical movement was of course at the center of the span, the amount decreasing toward the towers at which points there was none. The center of the ropes was 6 ft. above the center of the main pins on top of the towers. The chain hoists were to be originally set so that the eye-bar pins would be a uniform distance of 6 ft. below the ropes when the eye-bars were first lifted into position. The problem therefore was twofold:

First.—To make the ropes short enough at the start so that when they had elongated under the weight of all the bars, the sag at the center would be such as to permit all the eye-bars to be placed. If the sag reached 116 ft. before all the eye-bars were placed it would be difficult to slip the remaining ones over the pins.

Second.—To make the ropes long enough at the start so that when they again supported their own weight only (after they had been relieved of the weight of the eye-bars and had received their permanent set), the sag would still come within the limits of the amount of chain provided for the operation of the chain hoists. Otherwise, it would be impossible to swing the eye-bar cables by the chain hoists alone.

Tests of Erection Rope

The success of the operation therefore depended on the movements and behavior of the erection ropes. As this method of erection of eye-bars was a new departure in bridge construction it was decided to make a series of tests on the actual ropes used. In conferences with the engineers of the

American Steel and Wire Company which fabricated the ropes, the following procedure was adopted. The distance, center to center of the girders that served to anchor the ropes, was about 1 700 ft., and to provide for proper end fastening each of the twenty-four ropes was made 1 800 ft. long. It was very desirable to have the ropes stretch uniformly so that as far as possible each should carry its share of the load. Hence, the rope fabricators decided to spin four ropes in lengths of 10 800 ft. each, using extra precautions in the spinning, and then to cut each of these four pieces into six equal parts of 1 800 ft. each, thus making the twenty-four ropes.

One test piece was cut from each end of each of the four 10 800-ft. ropes. The twenty-four ropes were to be lettered from *A* to *X* so that the test pieces were cut from Ropes *A, F, G, K, M, R, S,* and *X*. The second long length of rope was spun only to about 9 000 ft., so that an additional 1 800-ft. piece had to be made. This became Rope *L,* from which a test piece was cut so that there were actually nine test pieces. These were fitted with sockets at each end, the distance between sockets being the standard testing length of about 36 in. The tests were made on the Emery hydraulic testing machine at the Worcester Plant of the American Steel and Wire Company, the measurements being taken by an extensometer reading to 0.0001 in. in a gauge length of 10 in.

The ropes were to be used twice in the field during the erection of the bars. They were first to be put up over the north columns. Supporting its own weight, each rope would be stressed to about 3 000 lb. The eye-bars for the north cable were then to be supported from the ropes, the average stress per rope under this condition being about 30 000 lb. The north cable would then be swung and the ropes again would support their own weight only, under a stress of 3 000 lb. per rope. Next the ropes were to be transferred to the tops of the south columns and the operation repeated.

To approximate erection conditions as far as possible each of the test pieces was stressed twice to an amount exceeding 30 000 lb. After taking a zero reading at 200 lb., loads were applied in 200-lb. increments up to 3 000 lb., and then by 1 000-lb. increments up to 35 000 lb. The load was then released and the tests repeated to 50 000 lb. This method applied for test pieces from Ropes *A, F, G,* and *K*.

For the remaining five ropes (*L, M, R, S,* and *X*), the following method which more closely approximated service conditions was adopted. After an initial load of 200 lb., increments of 200 lb. were applied up to 3 000 lb., and from there in increments of 1 000 lb., up to 35 000 lb. Instead of releasing the load entirely it was lessened gradually to 3 000 lb. and then increased a second time by 1 000-lb. increments to 50 000 lb. These test loads corresponded to the excessive stresses during the erection. They were carried to 35 000 lb. and to 50 000 lb. merely to have information in case the loads exceeded 30 000 lb. per rope. Actually, the stress in the back-stay portions was about 32 000 lb.

Readings of the elongations at each increment were plotted, making it possible to compute the moduli of elasticity for any stresses. From these

moduli could be determined the proper lengths for the ropes and the movements that would take place during the erection of the cables. These tests were made in August and September, 1924. Table 11 shows a representative set of gauge readings of elongations and Figs. 19 and 20 show the corresponding plots of these readings.

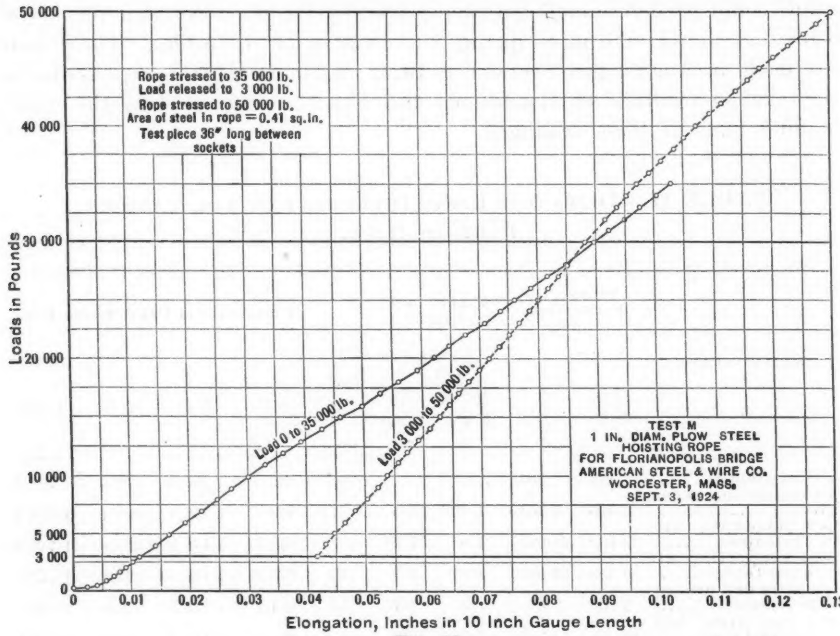
TABLE 11.—Distances Using Different, E, for Various Parts of Rope.

Conditions of load.	Stress, in Pounds per Rope.		Values of E, in Millions.		Sag, in feet.	Distances, in Feet, Along Rope.			
	Main span.	Approach.	Main span.	Approach.		AB.	BD.	DE.	Total.
Ropes on ground	200	200	....	....	....	272.9	1 136.5	292.6	1 702.0
Ropes supporting themselves	3 100	3 500	5.3	5.5	87	273.3	1 138.3	293.1	1 704.7
Ropes supporting half bars first time	18 600	21 000	7.3	7.5	107	274.7	1 143.9	294.6	1 713.2
Ropes supporting all bars first time	28 500	32 000	8.0	8.3	115	275.4	1 146.4	295.3	1 717.1
Ropes after first eye-bar cable released	2 700	3 000	1.5	1.7	99	274.1	1 141.7	293.9	1 709.7
Ropes supporting half bars second time	18 300	20 500	6.4	6.8	109	274.9	1 144.5	294.7	1 714.1
Ropes supporting all bars second time	28 500	32 000	8.0	8.4	115	275.4	1 146.4	295.3	1 717.1

Location of Erection Ropes

In order to save in the length of ropes required and to facilitate their transfer from the north to the south side, the rope girders for fastening the ends of the ropes were placed at Points A and E (Fig. 21) above the viaduct floor level. The permanent eye-bars from Points X to CA-6 on the continent side and from Points IA-6 to Y on the island side were erected on falsework bents near the pins at their permanent elevations. The rope girder pins were long and rested on the permanent eye-bar heads at Points A and E; two temporary eye-bars extended from the rope girder pins back to the permanent pins at Points CA-7 and IA-7. After the permanent eye-bars were placed from Points X to CA-6 and from Points Y to IA-6, the temporary eye-bars, temporary pins, and rope girders were erected on the north side of the bridge. The horizontal and vertical locations of the rope girder pins were thus known with respect to the pedestals in the anchorages and also to the bases of the main tower columns.

When the sag of the erection ropes was 115 ft. (when they held all the eye-bars, Fig. 22 (c)), the calculated distance, center to center of rope shoes, on the tops of the tower columns was 1 115 ft. 7 in. and the rope shoes were assumed to be exactly over the center of the tops of the main tower columns. Under this condition the length of Rope AE was 1 717.1 ft., made up of AB = 275.4 ft., BD = 1 146.4 ft., and DE = 295.3 ft., measured



Rope stressed to 35 000 lb.
Load released to 3 000 lb.
Rope stressed to 50 000 lb.
Area of steel in rope = 0.41 sq.in.
Test piece 36" long between sockets
Load 0 to 35 000 lb.
Load 3 000 to 50 000 lb.
TEST M
1 IN. DIAM. PLOW STEEL
HOISTING ROPE
FOR FLORIANOPOLIS BRIDGE
AMERICAN STEEL & WIRE CO.
WORCESTER, MASS.
SEPT. 3, 1924
Loads in Pounds
50 000
40 000
30 000
20 000
10 000
5 000
3 000
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
0.10
0.11
0.12
0.13
Elongation, Inches in 10 Inch Gauge Length

FIG. 19.

Rope stressed to 35 000 lb.
Load released to 3 000 lb.
Rope stressed to 50 000 lb.
Area of steel in rope = 0.41 sq.in.
Test piece 36" long between sockets
Load 0 to 35 000 lb.
Load 3 000 to 50 000 lb.
TEST X
1 IN. DIAM. PLOW STEEL
HOISTING ROPE
FOR FLORIANOPOLIS BRIDGE
AMERICAN STEEL & WIRE CO.
WORCESTER, MASS.
SEPT. 5, 1924
Loads in Pounds
50 000
40 000
30 000
20 000
10 000
5 000
3 000
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
0.10
0.11
0.12
0.13
Elongation, Inches in 10 Inch Gauge Length

FIG. 20.

along the ropes. Sections AB and DE had stresses of 768 000 lb., or about 32 000 lb. per rope, while Section BD had stresses varying from 660 000 lb. at the center of the span to 720 000 lb. adjacent to the tops of the towers with an average of 680 000 lb., or about 28 000 lb. per rope. From the tests the average modulus of elasticity of the ropes, when stressed the first time to 28 000 lb., was found to be 8 000 000 and the first time to 32 000 lb., to be 8 300 000.

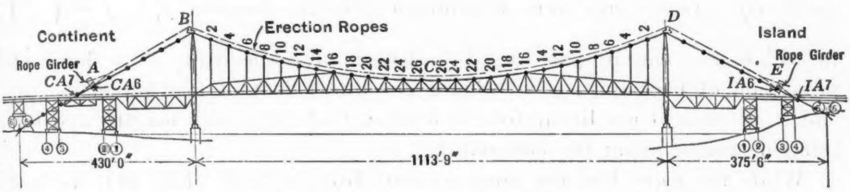


FIG. 21.—POSITION OF ERECTION ROPES.

These moduli were obtained by taking the average elongation of the test pieces at the various loads and substituting in the formula, $E = \frac{P\,L}{A\,\lambda}$, in which, P = the rope stress, in pounds; L = 10 in.; A = 0.41 sq. in., the area of a 1-in. rope; and λ = the elongation, in inches. Using the moduli of 8 000 000 and 8 300 000, the actual amounts of elongation of Sections AB, BD, and

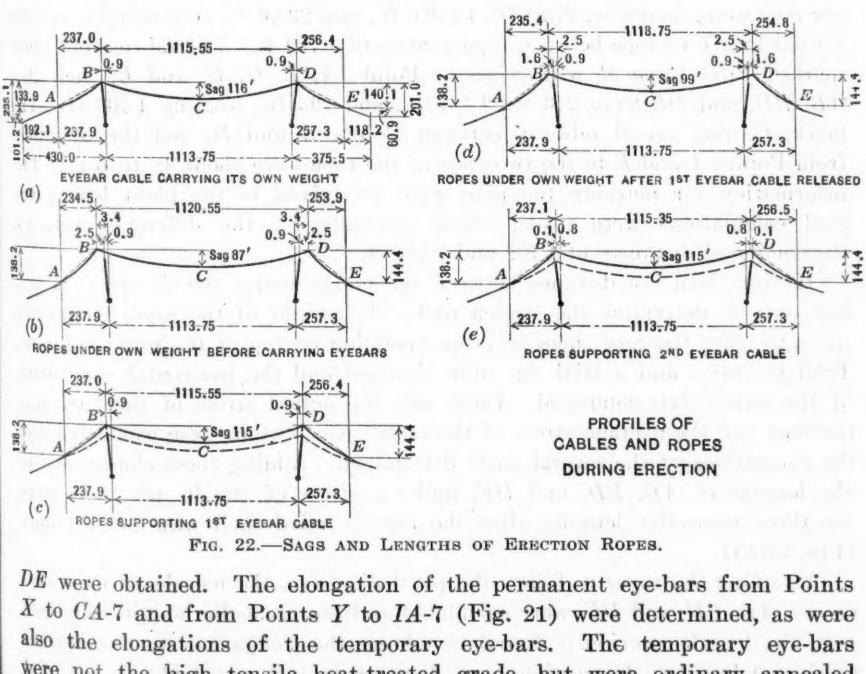


FIG. 22.—SAGS AND LENGTHS OF ERECTION ROPES.

DE were obtained. The elongation of the permanent eye-bars from Points X to CA-7 and from Points Y to IA-7 (Fig. 21) were determined, as were also the elongations of the temporary eye-bars. The temporary eye-bars were not the high tensile heat-treated grade, but were ordinary annealed eye-bars. For E a value of 27 000 000 was used for the heat-treated and

29 000 000 for the ordinary eye-bars. From these values and the known lengths the elongations of both the permanent and temporary eye-bars were ascertained.

Variation in Position of Ropes During Erection of Eye-bars

When the main span part of the ropes had a sag of 115 ft., the continent and island portions of the back-stays had sags of 5.4 ft. and 6.3 ft., respectively. These sags were determined from the formula, $f_1 = f \frac{w_1}{w} \left(\frac{l_1}{l}\right)^2$, in which, f = main span sag; l = length of main span; l_1 = horizontal projection of back-stay; w = weight per linear foot of load on main span; and w_1 = weight per linear foot of load on back-stay, both lengths and loads being measured along the horizontal.

While the ropes had the same sections from A to E (Fig. 21), the loads of the eye-bars were applied at points approximately 40 ft. apart along the ropes. These loads were not equal so that the ropes hung in an equilibrium polygon such that the horizontal components of the rope stress were constant from anchorage to anchorage. The general shape of the curve, however, approximated a parabola and the lengths of the rope sections were computed on that basis. These rope lengths under full load being AB = 275.4 ft., BD = 1146.4 ft., and DE = 293.3 ft., and the elongations being 2.6 ft., 9.8 ft., and 2.7 ft., the lengths of the three sections under a stress of 200 lb. per rope were, therefore, 272.8 ft., 1 136.6 ft., and 292.6 ft., respectively, making a total length of rope between rope girders of 1 702 ft. The fabricating plant marked Pilot Rope M with wires at Points A, B, C, D, and E, such that AB, BD, and DE were 273 ft., 1 137 ft., and 293 ft., totaling 1 703 ft. The mark, C, was placed midway between Points B and D, and the distances from Points A and E to the two ends of the rope were about 48 ft. 6 in. The information for marking the pilot rope was given to the plant before the final calculations were made, which accounts for the difference between the computed distances of 1 702 and 1 703 ft.

Having fixed the distance between the marks under 200 lb. stress it was necessary to determine the stretch under the weight of the ropes themselves so as to place the rope shoes in proper position on top of the tower columns. Trial positions and a trial sag were assumed and the horizontal component of the stress then computed. From this the actual stress of the back-stay portions and the average stress of the main span portion were calculated and the elongations of the several parts determined. Adding these elongations to the lengths of AB, BD, and DE, under a stress of 200 lb. per rope, gave the three respective lengths when the ropes carried their own weights only (Fig. 22(b)).

Assuming the ropes to follow the parabolic form, the lengths of these distances, AB, BD, and DE, were calculated and the parabolic lengths compared with the lengths previously found by adding the elongations to the original fabricated lengths. Corrections were then made and new trial sags, or trial positions of the rope shoes on top of the towers, or both, were assumed and

the process was repeated until the two sets checked within small amounts. It was found that the second or third trial gave the desired results sufficiently close. These figures: AB = 273.3 ft., BD = 1 138.3 ft., and DE = 293.1 ft., totaling 1 704.7 ft., with a sag of 87 ft., gave the length and sag of the ropes when stretched between the towers for the first time.

The same procedure was followed for the determination of the lengths and sags of the ropes after the north eye-bar cable was swung and the ropes were again supporting their own weight only (Fig. 22 (*d*)). This corresponded to the condition of Test Ropes *L, M, R, S,* and *X,* after they had been stressed up to 35 000 lb. and then released to 3 000 lb. These lengths were AB = 274.1 ft., BD = 1 141.7 ft., and DE = 293.9 ft., totaling 1 709.7 ft., with a sag of 99 ft.

The sags and lengths of the ropes when supporting the eye-bars a second time (Fig. 22 (*e*)) were then computed as previously. This corresponded to the condition of the test ropes when stressed a second time to 30 000 lb. per rope. For comparison the various sags under the several conditions are shown together in Fig. 22. Table 12 gives a summary of these sags, the moduli of elasticity, the stresses in the ropes, and the lengths of the ropes under the different conditions of loading.

When the ropes had been returned to the United States after having been used in the successful erection of the span, two pieces were cut from the pilot rope, *M,* one from the back-stay portion near *A* and the other from the main span portion near *B.* These were tested in February, 1926, at Worcester.

Shipment of Material

The steelwork was shipped in four sections. The advance shipment, consisting of the eye-bars, pins, and girders that were to be embedded in the masonry, was shipped in 1923. This material was placed by the general contractors during the construction of the anchorages. The three remaining shipments, comprising the remainder of the material, were made during the period from June to October, 1924; in the first was included both viaduct approaches and parts of the main towers; in the second the remainder of the towers, the eye-bars, pins, and parts of the main span; in the third and last, the remainder of the main span, the railing, erection ropes, and miscellaneous material. The eye-bars were shipped in nests of three bars each, making shipping pieces with over-all dimensions about 28 in. by 12 in. by 50 ft. long and weighing about 6 tons.

All the material arrived at the site in excellent condition, the eye-bars particularly standing the 6 000-mile ocean voyage without any damage other than slight rust in some spots where the shop coat of paint had worn off in transit.

Field Organization

On account of the special nature of the erection, the field organization was selected with the greatest care. It was considered to be better equipped for foreign work than any other ever assembled by the United States Steel Products Company.

TABLE 12.—GAUGE READINGS FOR TEST ROPES *M* AND *X*.

Load, in pounds.	Rope *M*.				Rope *X*.			
	First Test.		Second Test.		First Test.		Second Test.	
	Gauge reading, in inches.	Value of *E*.	Gauge reading, in inches.	Value of *E*.	Gauge reading, in inches.	Value of *E*.	Gauge reading, in inches.	Value of *E*.
200	0	....	......	....	0	....	......	....
400	0.0027	....	......	....	0.0010	....	......	....
600	0.0042	....	......	....	0.0017	....	......	....
800	0.0051	....	......	....	0.0023	....	......	....
1 000	0.0059	4 140 000	......	....	0.0029	8 400 000	......	....
1 200	0.0066	....	......	....	0.0035	....	......	....
1 400	0.0071	....	......	....	0.0040	....	......	....
1 600	0.0077	....	......	....	0.0047	....	......	....
1 800	0.0083	....	......	....	0.0053	....	......	....
2 000	0.0088	5 550 000	......	....	0.0060	8 120 000	......	....
2 200	0.0094	....	......	....	0.0067	....	......	....
2 400	0.0099	....	......	....	0.073	....	......	....
2 600	0.0104	....	......	....	0.081	....	......	....
2 800	0.0110	....	......	....	0.088	....	......	....
3 000	0.0115	6 360 000	0.0418	1 750 000	0.0094	7 780 000	0.0395	1 850 000
4 000	0.0141	6 930 000	0.0431	2 260 000	0.0125	7 820 000	0.0414	2 360 000
5 000	0.0166	7 350 000	0.0448	2 720 000	0.0157	7 780 000	0.0433	2 820 000
6 000	0.0193	....	0.0466	....	0.0191	....	0.0454	....
7 000	0.0222	....	0.0483	....	0.0224	....	0.0475	....
8 000	0.0247	....	0.0503	....	0.0254	....	0.0496	....
9 000	0.0272	....	0.0521	....	0.0288	....	0.0518	....
10 000	0.0301	8 100 000	0.0539	4 530 000	0.0321	7 600 000	0.0537	4 550 000
11 000	0.0331	....	0.0556	....	0.0351	....	0.0556	....
12 000	0.0361	....	0.0573	....	0.0384	....	0.0576	....
13 000	0.0391	....	0.0592	....	0.0415	....	0.0596	....
14 000	0.0427	....	0.0611	....	0.0445	....	0.0615	....
15 000	0.0462	7 920 000	0.0629	5 830 000	0.0475	7 700 000	0.0633	5 800 000
16 000	0.0499	....	0.0646	....	0.0505	....	0.0650	....
17 000	0.0527	....	0.0662	....	0.0532	....	0.0668	....
18 000	0.0560	....	0.0680	....	0.0560	....	0.0686	....
19 000	0.0594	....	0.0698	....	0.0585	....	0.0705	....
20 000	0.0621	7 860 000	0.0716	6 820 000	0.0613	7 950 000	0.0720	6 690 000
21 000	0.0646	....	0.0735	....	0.0639	....	0.0737	....
22 000	0.0675	....	0.0753	....	0.0662	....	0.0754	....
23 000	0.0710	....	0.0770	....	0.0686	....	0.0770	....
24 000	0.0737	....	0.0786	....	0.0710	....	0.0788	....
25 000	0.0762	8 000 000	0.0803	7 600 000	0.0732	8 350 000	0.0805	7 580 000
26 000	0.0790	....	0.0820	....	0.0753	....	0.0820	....
27 000	0.0817	....	0.0838	....	0.0775	....	0.0835	....
28 000	0.0849	....	0.0852	....	0.0797	....	0.0852	....
29 000	0.0870	....	0.0868	....	0.0821	....	0.0868	....
30 000	......	8 160 000	0.0884	8 280 000	0.0841	8 720 000	0.0886	8 260 000
31 000	0.0923	....	0.0901	....	0.0861	....	0.0903	....
32 000	0.0950	....	0.0918	....	0.0884	....	0.0921	....
33 000	0.0977	....	0.0937	....	0.0905	....	0.0938	....
34 000	0.1004	....	0.0954	....	0.0926	....	0.0954	....
35 000	0.1030	8 280 000	0.0971	8 800 000	0.0947	9 040 000	0.0972	8 800 000
36 000	......	....	0.0992	....	......	....	0.0991	....
37 000	......	....	0.1014	....	......	....	0.1013	....
38 000	......	....	0.1036	....	......	....	0.1032	....
39 000	......	....	0.1053	....	......	....	0.1051	....
40 000	......	....	0.1073	9 080 000	......	....	0.1076	9 080 000
41 000	......	....	0.1096	....	......	....	0.1096	....
42 000	......	....	0.1119	....	......	....	0.1118	....
43 000	......	....	0.1141	....	......	....	0.1136	....
44 000	......	....	0.1162	....	......	....	0.1156	....
45 000	......	....	0.1183	9 260 000	......	....	0.1178	9 320 000
46 000	......	....	0.1208	....	......	....	0.1199	....
47 000	......	....	0.1233	....	......	....	0.1222	....
48 000	......	....	0.1257	....	......	....	0.1245	....
49 000	......	....	0.1282	....	......	....	0.1266	....
50 000	......	....	0.1307	9 350 000	......	....	0.1293	9 430 000

The whole scheme was a departure from the conventional method of suspension bridge erection, in the nature of an experiment 6 000 miles from office and plant, so that every precaution was taken to provide against possible contingencies. The superintendent was assisted by two foremen, seventeen American bridge men, and a time-keeper. Two field engineers were assigned to the field organization. In addition to the American nuclei of twenty-two men, the field force was augmented by local labor. This Bazilian labor did most of the work in the material yard while the Americans attended to the actual raising of the steel.

Several of the natives, however, developed into fairly good bridge men and were used on the erection of the main span. The Company was fortunate in obtaining the local services of two Italians who were familiar with steel erection and who were put in charge of the native labor in the material yard.

Unloading of Material

The first contingent of the erection organization left New York, on July 5, 1924, and arrived at Florianopolis about July 25. This part of the organization distributed the steel over the material yard, which was located at the mainland end of the bridge, as the several steel shipments arrived. The large, ocean-going freight boats could not enter the shallow strait between the Island of Santa Catharina and the mainland, and it was necessary, therefore, to discharge the cargo from the ships into lighters and tow the lighters about 10 miles from the ships to the landing dock.

The locomotive crane in the yard unloaded the lighter during the day and distributed the material over the yard. Usually, during the night the lighter was towed back to the ship, reloaded with another batch of material, and towed back to the dock, ready for unloading the following morning, although the barges were often unloaded also at night.

Erection of Approach Viaducts

Erection of steel was started on the continent end of the bridge by erecting Towers 1-2 and 3-4 (Fig. 21), using a locomotive crane. A "jinniwink" derrick was then raised to the top of Tower 3-4. The two trusses of the 110-ft. span, Tower 2-3, were riveted up on the ground and then raised into positions. In the meantime falsework had been driven under the 185-ft. span (adjoining the tower), which was erected on the falsework by the jinniwink. The erection of the remainder of the continent wiaduct was completed by the jinniwink, working back toward the west abutment. Using a track laid in the material yard parallel to the continent viaduct on the south side, the locomotive crane carried the viaduct steel to its proper location, from which it was raised to the viaduct floor level by the jinniwink.

The east or island viaduct was erected in the same general manner except that the steel had to be lightered over to the island from the material yard, and that there was no locomotive crane on the island side.

No special difficulties were encountered in the erection of the viaducts except that the rate of progress was somewhat slow owing to the large number of pieces to be handled.

The continent viaduct was completed about November 8, 1924, taking 3 months; and the island viaduct was completed about January 5, 1925, taking $2\frac{1}{2}$ months.

Erection of Main Towers

The shoes of the continent tower were set in the center of the continent piers using the jinniwink derrick on top of the viaduct. The first sections of the continent tower columns (Fig. 23) were placed tilted back toward the viaduct so that the tops of the columns would come about 1 ft. back of their final position making the distance, center to center of column tops, 1 115 ft. 7 in. (for reasons already explained). The temporary struts attaching the tower to the 185 ft. span, during erection are shown in Fig. 24. The lower bracing and the main cross-girder were placed as the column sections were erected. This much of the tower was erected using the jinniwink on the deck of the viaduct.

A special climbing traveler (Fig. 25) was then built around the tower and the remaining six column sections were placed by this creeper traveler, which climbed up the tower as the sections were added. The remainder of the tower bracing was placed as the column sections were added.

As a preliminary measurement across the channel had indicated that the distance, center to center of piers, might be 6 in. in error, these island piers were made 2 ft. larger in diameter than the continent piers. When the island piers were completed sufficiently to enable a more accurate measurement to be taken, it was found that the centers of the continent and island piers were 3 in. farther apart than the proper distance of 1 113 ft. 9 in. The island tower shoes, therefore, were set 3 in. off the center of the island piers in accordance with the triangulation measurements so that the steel towers would be the proper distance, center to center. With the island piers 2 ft. larger in diameter than the continent piers, the minimum distance from the main shoes to the edge of the island piers was at least as great as the same distance on the continent piers.

The island main tower was then erected in a manner similar to the continent tower (Fig. 26). The material for both towers was loaded on the lighter and brought to the foot of the towers. The creeper traveler performed its part of the work in a very satisfactory manner. The continent tower was completed about December 27, 1924, taking 6 weeks; and the island tower was completed about February 1, 1925, taking 5 weeks.

Erection of Ropes, Trolleys, Clamps, and Chain Hoists

The steel towers were erected by February 1, 1925. The beam grillages, temporary rollers, and rope shoes were then placed on top of the two north columns. All the eye-bars from the anchorages up to the viaduct floor level had been placed previously, and the temporary eye-bars and the rope girders on the north side of the bridge followed during the first week of February. A continuous $\frac{3}{4}$-in. rope was then run across the main channel, lifted to the top of each tower, and placed on the top transverse tower strut adjacent to the north eye-bar shoes.

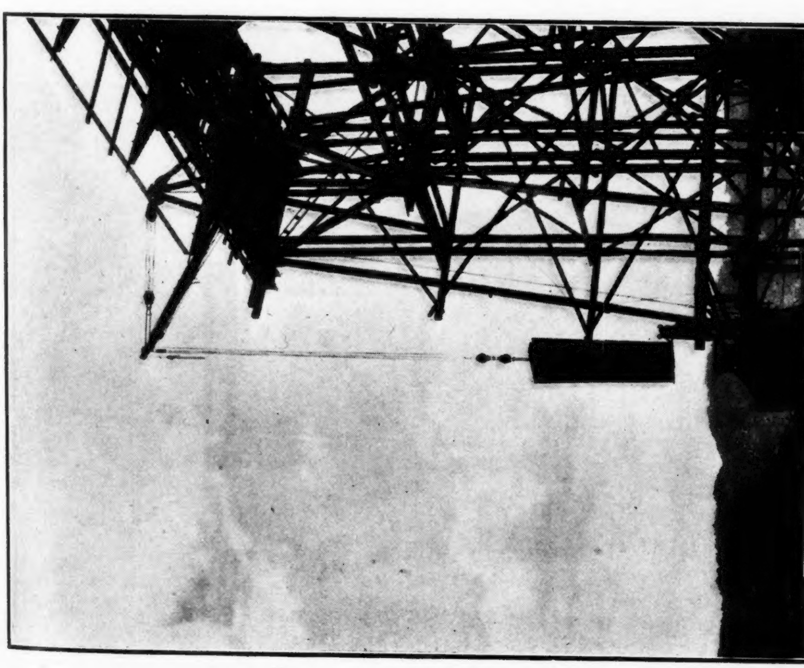


FIG. 23.—VIEW SHOWING ERECTION OF FIRST SECTION OF CONTINENT COLUMNS, FLORIANOPOLIS BRIDGE.

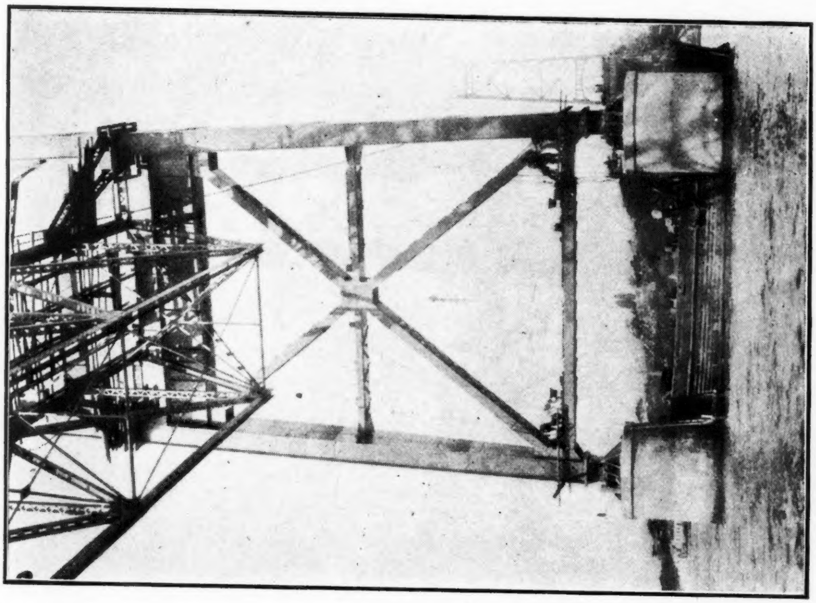


FIG. 24.—VIEW OF LOWER PART OF CONTINENT TOWER, FLORIANOPOLIS BRIDGE, SHOWING TEMPORARY STRUT CONNECTIONS.

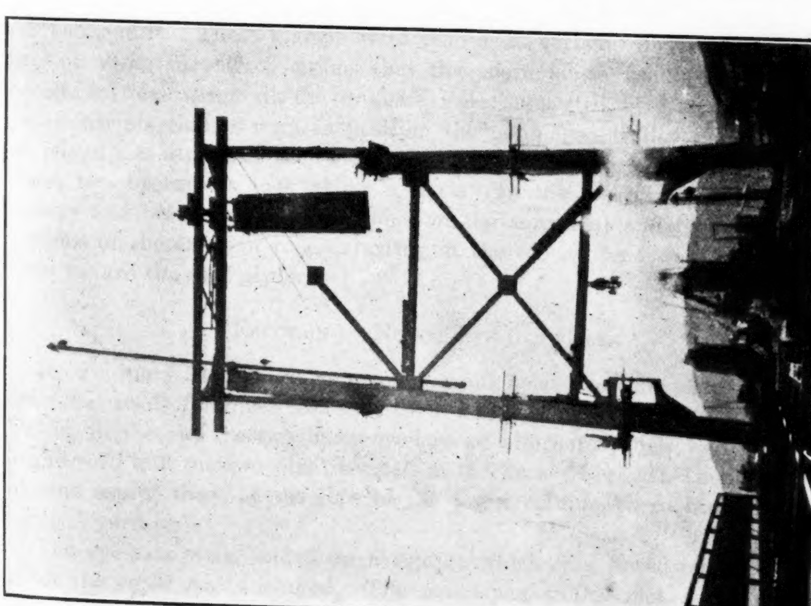


FIG. 25.—VIEW SHOWING ERECTION OF EIGHTH SECTION OF CONTINENT TOWER COLUMN, FLORIANOPOLIS BRIDGE.

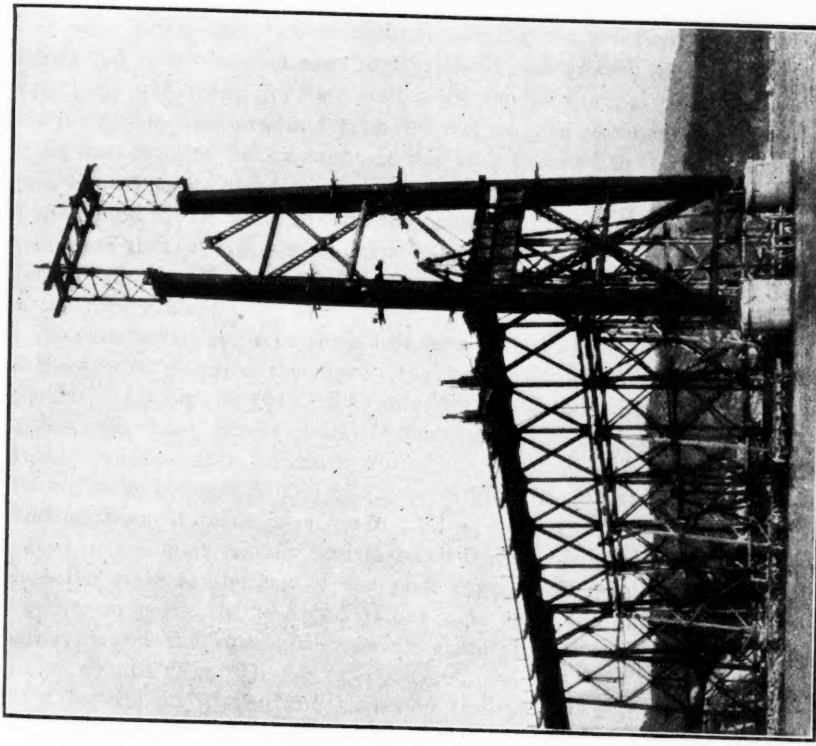


FIG. 26.—CREEPER TRAVELER ON ISLAND FOR TOWER ERECTION, FLORIANOPOLIS BRIDGE.

Twelve of the twenty-four reels containing the erection ropes had been transferred from the continent to the island side and on February 10, 1925, everything was ready for the placing of the pilot rope. The free end of this pilot rope was unwound from its reel on the continent side and lifted to the floor level of the viaduct. It was then fastened to the continuous $\frac{3}{4}$-in. rope, hauled up to the top of the tower, across the strait and over the top of the island tower, using the hoisting engine on the island viaduct. The two ends were then wound one and one-half times around the rope girders and clamped into position so that Marks *A* and *E* came at the proper position on the rope girders.

The centers of the rope shoes had been placed $2\frac{1}{2}$ ft. back from the center of the eye-bar shoes on the tops of the main columns in accordance with the calculations. (Fig. 22 (*b*)). The pilot rope was then lifted into the grooves on the rope shoes, Marks *B* and *D* being placed in the center of these shoes. An observation taken on Mark *C* midway between Marks *B* and *D* showed the sag to be between 86 and 87 ft., or within 1 ft. of the calculated distance. During the next three days the remaining twenty-three erection ropes were placed in a similar manner and fastened to the rope girders so that the sags were the same as the sag of the pilot rope. The rope ends were fastened together in pairs, the free end of one rope after one and one-half complete turns around the rope girder being clamped to its adjacent neighbor by 6-rope clamps (Fig 27).

After the main span and back-stay trolleys had been placed in position on the ropes, the main span trolley was moved across from the island to the continent side placing the clamps which bound the twenty-four ropes into one cable unit. These clamps were placed at variable distances along the erection ropes in such positions that the chain hoists would hang approximately vertical when all the eye-bars were supported by the ropes. After the twenty-six clamps were in position the main-span trolley moved back to the island side attaching the twenty-six Harrington chain hoists to the clamps. These two operations took about a week. In the meantime, the back-stay trolleys had been placing the clamps on the continent and island back-stay portions of the erection ropes, starting at the top of the towers and working down toward the rope girders.

Erection of North Eye-Bar Cable

By February 23 all the clamps and chain hoists were in place and everything was ready for the erection of the eye-bars. To eliminate the driving of pins up in the air, the two inner eye-bars of alternate panels were assembled in the yard and the two pins inserted in the holes (Fig. 28). In this manner all pins except those at the tops of the tower columns were inserted in the material yard.

The eye-bars were loaded on a lighter which was towed out to position under the ropes and anchored. The main-span trolley moved to a position over the lighter, picked up the two inner eye-bars with pins for Panel 24-26,

Island Side, raised them up into position at the proper chain hoists where they were fastened to the lower ends of the chain hoists approximately 6 ft. below the erection ropes. The trolley then lifted the eye-bars, 24-26, Continent Side, into position. Fig. 28 is a near view of a pair of eye-bars leaving the barge, while Fig. 31 shows the trolley lifting the second pair of eye-bars, the first pair having already been attached to the chain hoists.

On the main span the ropes were actually about 16 ft. longer than the chord between the tops of the columns while on the back-stays the arc was only about 2 in. longer than the chord. It was impossible, therefore, to pull the shoes off the towers by loading the main span; but it was possible for them to slip off backward if the back-stay eye-bars had been placed with few or no eye-bars weighting the main span.

At the start, therefore, the eye-bars were grouped toward the center of the span. The object was to put as much weight as possible on the central part of the main-span erection ropes so as to bring the rope shoes from 2½ ft. back of the center of the columns (Fig. 29) to a position over the center of the columns (Fig. 30) as quickly as possible. By the time twenty-six eye-bars had been supported on the erection ropes, the rope shoes had moved from a position 2½ ft. away from the column center to 1 ft. from the column center. It was then felt safe to commence the erection of the eye-bars on the two back-stays.

When all the eye-bars had been placed the temporary rope shoes had moved from the original position of 2 ft. 6 in. back of the saddle-casting to a position practically over the center of the saddle (Fig. 30). The entire erection of the 156 bars took 2 weeks.

By Saturday noon, March 7, all the eye-bars were erected, the entire eye-bar cable hanging from the erection ropes. The day being clear and calm, it was decided to swing the eye-bar cable that afternoon. The actual sag at that time when the erection ropes carried all the eye-bars was 113 instead of 115 ft., the difference being partly due to not placing the gusset-plates and hangers as was originally contemplated when the calculations were made. The eye-bars had been slipped over the pins without difficulty, thanks to the elongated pin-holes and also to the use of short pilot nuts on each pin. (Fig. 28).

Men took positions on the eye-bars at the chain hoists and at 2:30 P. M. the signal was given for the men to slacken off the chain hoists; and in a few minutes the normal 6-ft. gap between the erection ropes and eye-bars began to increase. Some of the men operated the chain hoists faster than others and to keep the eye-bar cable in a smooth curve it was necessary to have them wait until the slower ones could catch up. By 3:15 P. M. all the chain hoists were slack, showing that the eye-bar cable was swinging free of the ropes under its own weight and the erection ropes were carrying their own weight under a sag of between 99 and 100 ft. instead of the calculated sag of 99 ft. The entire operation of swinging took 45 min. and was a complete success in every respect.

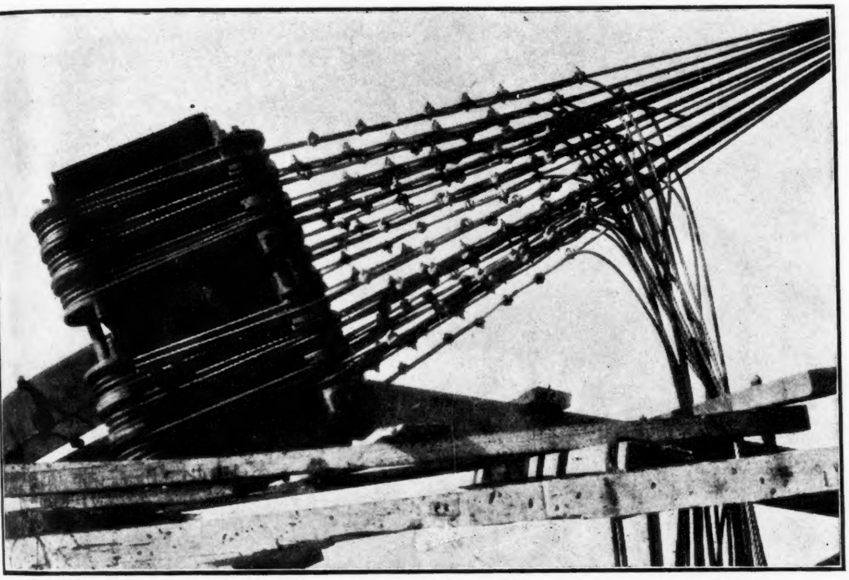


FIG. 27.—CLAMPING OF TWENTY-FOUR ERECTION ROPES AROUND ROPE GIRDER, FLORIANOPOLIS BRIDGE.

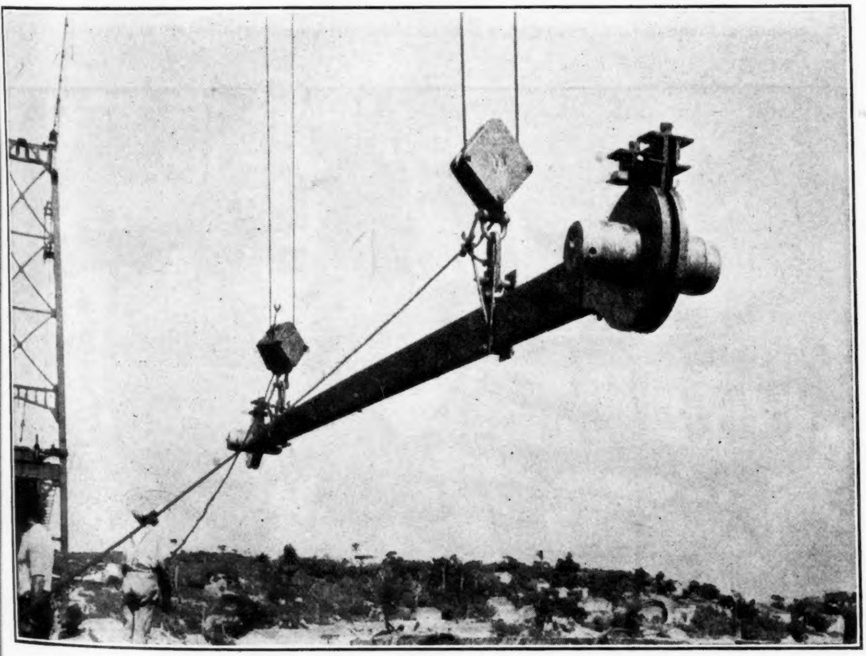


FIG. 28.—VIEW SHOWING RAISING OF EYE-BARS WITH PINS, FLORIANOPOLIS BRIDGE.

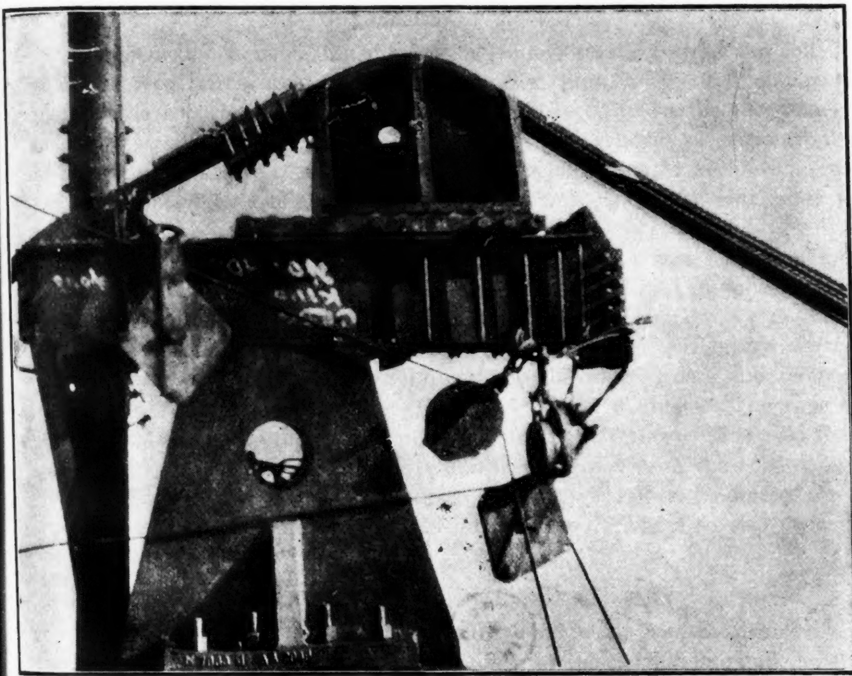


FIG. 29.—ERECTION ROPE SHOES BEFORE LOADING THE ROPES, FLORIANOPOLIS BRIDGE.

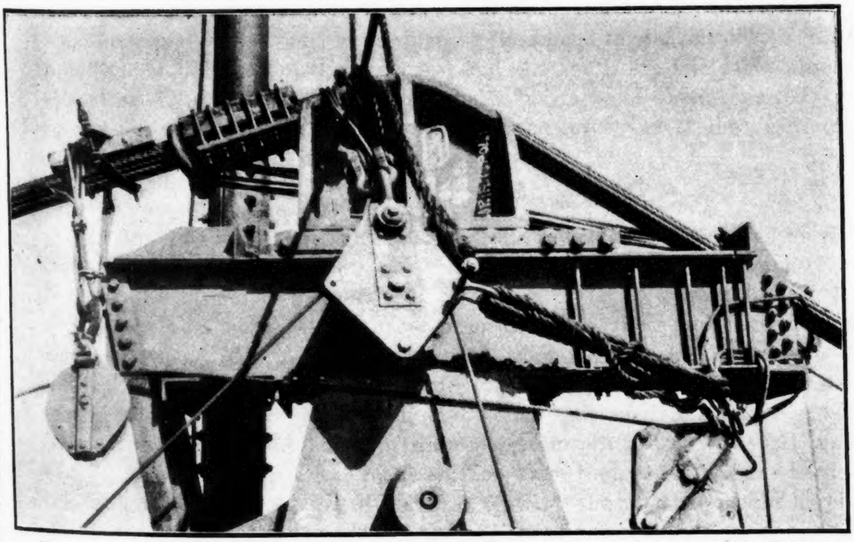


FIG. 30.—ROPE SHOE OVER CENTER OF COLUMN, ROPE CARRYING ALL THE EYE-BARS, FLORIANOPOLIS BRIDGE.

Transfer of Ropes

As the next operation, the main-span trolley, starting from the continent tower, moved across the ropes removing the chain hoists and clamps and placing the rope hangers and steel hangers (Fig. 34). In the meantime the back-stay trolleys were removing the clamps and eye-bar supports from the back-stay parts of the ropes. These operations took about a week. The main-span and back-stay trolleys were taken down and the twenty-four ropes were moved from the north to the south of the towers.

The ropes were first lifted out of the rope shoes one at a time and spread out across the top transverse struts of the main towers. The ends were then unclamped, slipped off the rope girders, and fastened temporarily to the viaduct floor-beams. The rope shoes, rollers, and I-beam grillages were transferred to the tops of the south columns, and the rope girders and temporary eye-bars were transferred to the south sides of the viaducts. The rope ends were then fastened to the rope girders in a manner similar to that used for the north cable, and the ropes were lifted into the grooves on the rope shoes on top of the tower columns. The three trolleys were then re-erected on the ropes and the rope clamps and chain hoists placed. These several operations took about two weeks.

Erection of South Eye-Bar Cable

On March 26, erection of the south eye-bar cable was commenced. The method used here was the same as that used for the north cable and the entire 156 eye-bars were placed in one week, just one-half the time that it took for the north cable.

By April 2 all the eye-bars were hanging from the erection ropes. At 3:30 P. M. the signal was given for the men at the hoists to slacken off, (Fig. 32); by 4:00 P. M. all the hoists were loose and the south eye-bar cable was self-supporting (Fig. 33). Clamps and chain hoists were then removed; rope hangers and steel hangers were erected; and the three rope trolleys were taken down. These operations took about four days.

Erection of Trusses

The first two truss panels at each end of the bridge were erected by jinniwinks standing on the viaducts, the remainder of the main span being erected by the overhead trolley method.

Two erection ropes were lifted from the rope shoe and placed on the wheels on top of the transverse struts adjacent to the cast-steel shoes. The ends of these ropes were then loosened from the rope girders and fastened around the rope-girder pins, each end being shortened 5 ft. so that when fastened to the pins the length of the ropes from pin to pin was 1 691 instead of 1 701 ft. This change was made so that when the center bottom chord section weighing 18 000 lb. was lifted into position the member would be above its proper elevation and could be easily slipped into position and connected to the bottom of the steel hangers.

By this time the verticals of the trusses had already been placed (Fig. 34). Before any other permanent member was erected a test load of 18 000 lb. was

placed on the small rope trolley that had been erected on the two 1-in. ropes, and the trolley was pulled out to the center of the span. This test checked the calculations and proved that it was possible to erect the bottom chords by this method. It would have been a simple matter to have made the ropes shorter; then the trolley would have been high enough to attach the truss member, but the stress per rope would have exceeded the 32 000 lb., and it was undesirable to use three ropes as the trolley equipment had been designed for support on two ropes only.

The south bottom chords were the first truss members erected by the overhead method. These were placed starting from the continent and working toward the island. When the continent half of the bottom chords was suspended from the hangers the weight distorted the eye-bar cable. As the remainder of the south bottom chord was placed the eye-bar cable gradually came back to its symmetrical form (Fig. 35); and when the last section of bottom chord was placed there was a gap of about 6 in., due to the camber being high under the partial dead load. The south truss diagonals were then erected, using the same trolley. The bottom chords were erected in 14 hours and the diagonals in a similar period.

Another trolley was then erected on two more of the erection ropes placed adjacent to the cast-steel shoes on the north columns and the north bottom chords and diagonals erected in a similar manner. The bottom chords of this truss were placed in 10 hours and the diagonals also in 10 hours.

The north and south top chords were then erected, first on the continent side and then on the island side. This operation took about 18 hours. Fig. 36 shows a top chord being lifted into position. Erection of the floor-beams, stringers, and bottom laterals then took place, followed by the top laterals, top struts, and portals. The total time consumed in the main span erection was 45 working days for the erection of about 1 000 members. Fig. 37 shows a view from the continent tower looking toward the island during the erection of the trusses.

Placing Temporary Counterweight and Drilling Holes in Web Members

When all the steel was erected the total dead load was only 3 000 lb. per lin. ft. so that the sag had not reached 120 ft. Under this condition it was impossible to connect the web members and top chords because the fabricated lengths of these members were based on the geometric lengths they would occupy under a sag of 120 ft.

The delivery date of the floor lumber was uncertain so that some temporary load had to be placed upon the main span to bring the eye-bar cable to the correct dead load position, in order that the holes in the blank gusset-plates could be drilled and the diagonals outside Panel Points 16 could be connected under zero stress. As there was considerable sand on the beach along the continent shore it was decided to make use of this material. A sand load equivalent to 1 400 lb. per lin. ft. of bridge was placed on the steel floor system, and this brought the diagonals from Points 0 to 16 at each end of the bridge to such position that connections could be made with

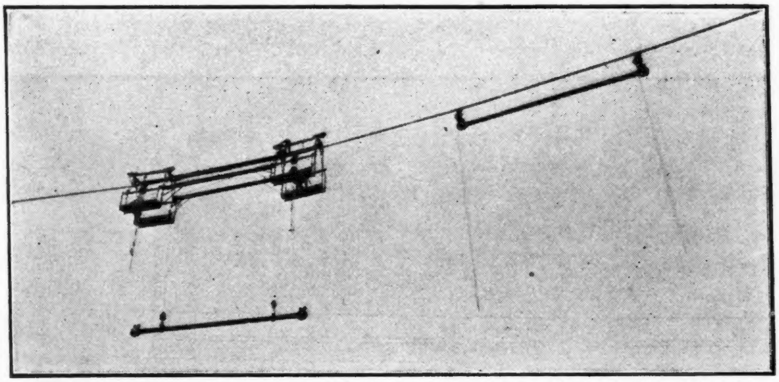


FIG. 31.—TROLLEY LIFTING SECOND PAIR OF EYE-BARS, FLORIANOPOLIS BRIDGE.

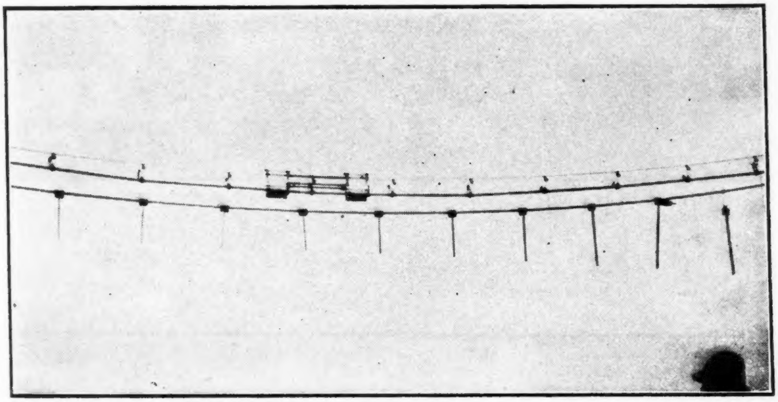


FIG. 32.—SOUTH EYE-BAR CABLE SUPPORTED FROM ROPES, 3:30 P. M., APRIL 2, 1925, FLORIANOPOLIS BRIDGE.

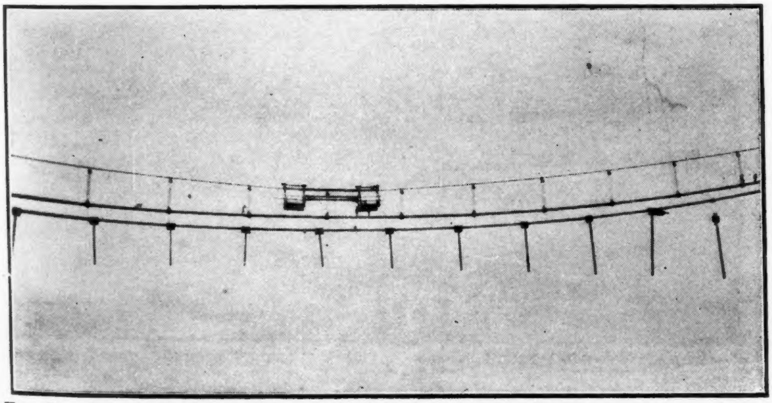


FIG. 33.—SOUTH EYE-BAR CABLE, SELF-SUPPORTING AND FREE OF ROPES, 4:00 P. M., APRIL 2, 1925, FLORIANOPOLIS BRIDGE.

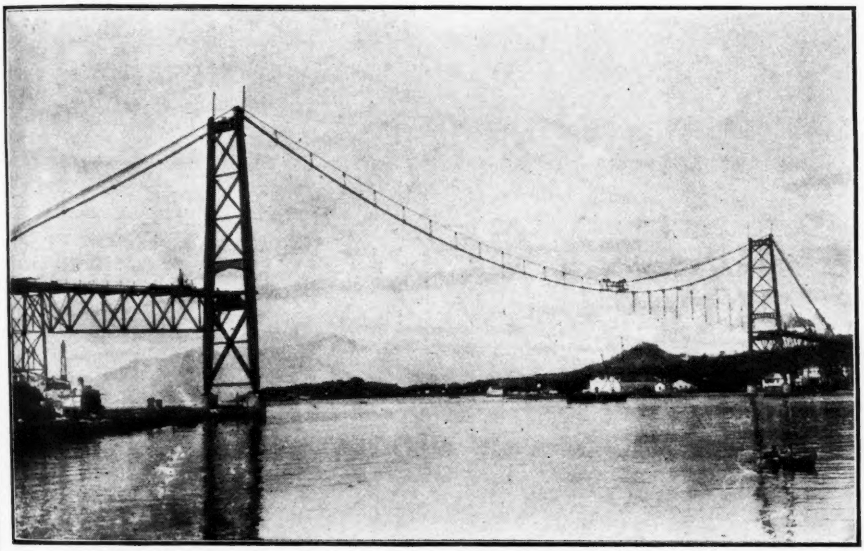


FIG. 34.—FLORIANOPOLIS BRIDGE: TROLLEY REMOVING CLAMPS AND HOISTS AND PLACING OF HANGERS.

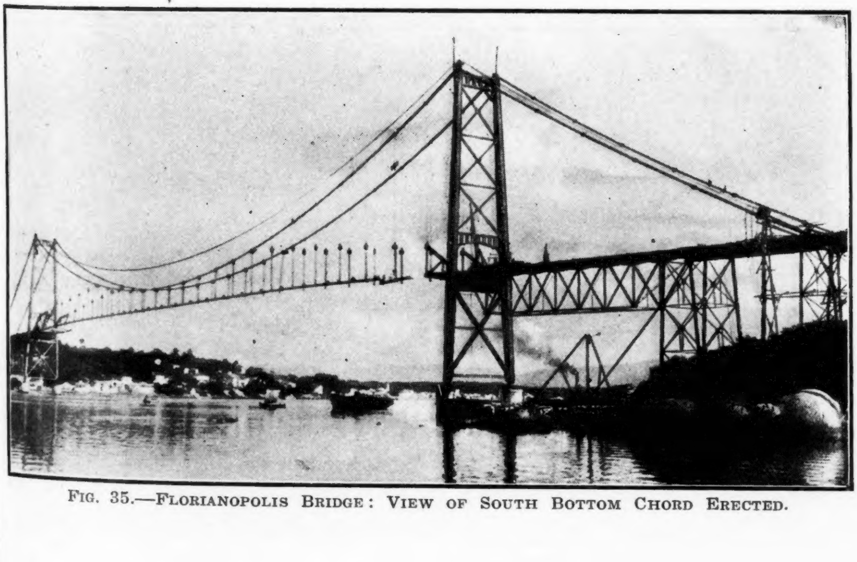


FIG. 35.—FLORIANOPOLIS BRIDGE: VIEW OF SOUTH BOTTOM CHORD ERECTED.

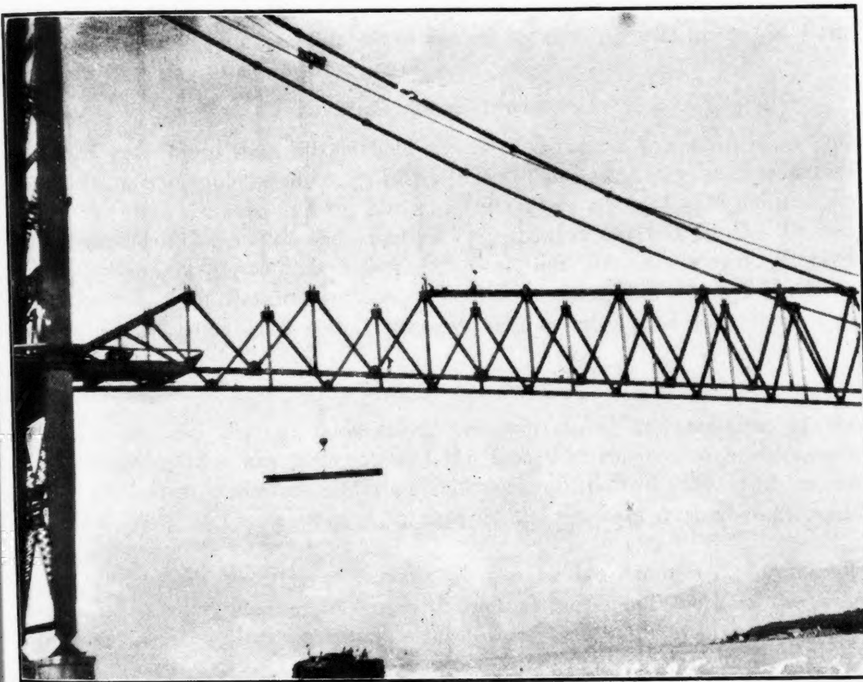


FIG. 36.—VIEW SHOWING ERECTION OF TOP CHORD, FLORIANOPOLIS BRIDGE.

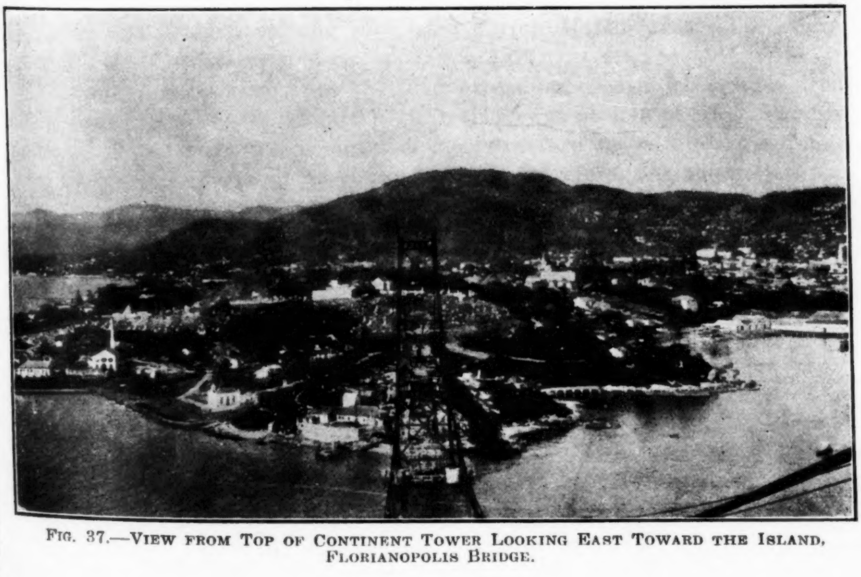


FIG. 37.—VIEW FROM TOP OF CONTINENT TOWER LOOKING EAST TOWARD THE ISLAND, FLORIANOPOLIS BRIDGE.

practically no drifting. The 1 728 holes in the bottom chord gusset-plates between Points 16 were then drilled and the diagonals connected under this condition. One of the erectors set a record by drilling 240 holes (in ½-in. material) in 1 day of 8 working hours.

Riveting and Painting

There were more than 50 000 field rivets to drive on the main span and this work was completed during July, 1925. The temporary sand counterweight was then removed and the structure given two coats of field paint.

The last contingent of the erection organization arrived in the United States on August 31, so that a few days less than fourteen months elapsed from the time that the first section of field forces left the country until the last section returned; the actual time of field erection was just about one year.

Summary of Erection

The successful design, fabrication, transportation, and erection of the Florianopolis Bridge has demonstrated the fact that suspension bridges with eye-bar cables are practical and that falsework of the ordinary kind, or, on the other hand, elaborate staging to support the eye-bars during erection, is unnecessary.

A particularly gratifying feature of the bridge from an engineering standpoint was the manner in which the actual movements both of the erection ropes and of the permanent steel towers and main span agreed with the designed movements. It was possible from the calculations to predict the movements or positions under various conditions of erection with a very small error.

The placing of the wooden floor was done by the general contractors, Byington and Sundstrom, and completed during March, 1926. The official opening of the bridge took place on May 3, 1926.

During the entire work in the field extending over a whole year from about August 1, 1924, to August 1, 1925, there was no loss of life. Furthermore, the actual cost of the entire steel superstructure was within the estimate. The Florianopolis type of suspension bridge, therefore, has shown itself to be a practical, a safe, and an economical type of structure.

Total Weights, Quantities and Costs

The weight of steel in the Florianopolis Bridge, including the approaches, is approximately 4 400 tons, made up as follows:

Chains:		
Eye-bars and pins	780	tons
Main Span:		
Trusses and bracing	840	"
Floor system	420	"
Main Towers:		
Columns and bracing	830	"
Castings	90	"

Anchorages:		
Eye-bars and girders	110	tons
Approaches:		
Spans (including floor and bracing)	960	"
Towers and bracing	290	"
Miscellaneous:		
Railings, etc.	80	"
Total	4 400	tons

The total quantity of concrete in the anchorages and piers is approximately 14 500 cu. yd., made up as follows:

Island anchorage	3 500	cu. yd.
Continent anchorage	6 000	" "
Piers and abutments	5 000	" "
Total	14 500	cu. yd.

The total cost of the Florianopolis Bridge, including the contractor's profit, as represented by the amount of the general contract, was upward of $1 400 000 (the exact amount is difficult to state on account of the fluctuating value of the Brazilian currency at the time). Of this total amount, the cost of the superstructure, as represented by the amount of that sub-contract, was approximately one-half.

ACKNOWLEDGMENTS

The Florianopolis Bridge was built for the Brazilian State of Santa Catharina under a general contract awarded to Byington and Sundstrom, of São Paulo. That firm handled the substructure work, including the foundations, piers, anchorages, and abutments.

The principal contractors retained Messrs. Robinson and Steinman, of New York, as Consulting and Designing Engineers, and L. N. Gross as Associated Consulting Engineer on construction, plant, and equipment. G. A. Brinkerhoff, Assoc. M. Am. Soc. C. E., went to Florianopolis to superintend the carrying out of the Consulting Engineers' plans for the foundations and masonry. W. E. Joyce, M. Am. Soc. C. E., was in charge of the office organization of Robinson and Steinman during the preparation of the final design. J. Gunnar, A. Johnson, J. London and R. Boblow, Juniors, Am. Soc. C. E., contributed in the preparation of the design.

The steel superstructure was fabricated by the American Bridge Company, C. W. Bryan, M. Am. Soc. C. E., Chief Engineer, and erected by the United States Steel Products Company, W. H. Stratton, Manager, Bridge Department. The development of the design was done in the Eastern Division of the American Bridge Company, J. E. Wadsworth, M. Am. Soc. C. E., Division Engineer, S. J. Ott, M. Am. Soc. C. E., Assistant Engineer. The detail drawings were made at the Trenton Plant, J. E. Elliot, Engineer. The eyebars, pins, and large castings were fabricated at the Ambridge Plant and the remainder of the material at the Elmira Plant.

The wire rope was manufactured at the Worcester Plant of the American Steel and Wire Company, J. F. Howe, Engineer, the railing by the Somerville Company, and the hand-operated chain hoists by the Harrington Hoist Company. The shop paint was Dixon's red lead graphite primer and the field paint was Dixon's silica graphite.

The field work was carried out under the direction of R. Khuen, Jr., M. Am. Soc. C. E., General Manager of Erection and C. S. Garner, Manager of Foreign Erection. Messrs. E. G. Amesbury was Resident Engineer and T. S. Melton was Field Superintendent.

Mr. Sundstrom, of the contracting firm, dividing his time between New York and Brazil, gave his personal attention to the preliminary surveys and studies, final designs, construction of the substructure, and the superstructure contract. Acknowledgment is due him for his good offices in the various negotiations with the local officials, which smoothed over many difficulties usually encountered in foreign work due to lack of familiarity with local customs and procedure.

The success of the structure was made possible by the experimental work on the method of heat-treating eye-bars, to get an elastic limit of 75 000 lb. per sq. in., developed by C. W. Bryan, Chief Engineer, and C. G. E. Larsson, M. Am. Soc. C. E., Assistant Chief Engineer, of the American Bridge Company. One of the writers, Mr. Grove, Assistant Engineer, was in charge of the engineering features of the steel superstructure and spent several months at Florianopolis during the erection of the eye-bar cables and of the stiffening trusses.

HEXAGONAL PLANNING, TRAFFIC INTERCEPTER, AND ORBIT*

By Noulan Cauchon,† Esq.

Synopsis

The problem discussed in this paper is congestion, the writer's aim being to present a study for the elimination of the problem.

Cause of Congestion

Status.—Congestion comes of ill-proportioned functions and inadequate services, nearly all variables from lack of definite purpose. Congestion comes of the lack of functional, organic "town planning," defined as "the scientific and orderly disposition of land and buildings in use and development, with a view to obviating congestion and securing economic and social efficiency, health and well-being in urban and rural communities." Congestion comes of inadequacy of living conditions due to shape and disposition of blocks, to over-density of population; of the scheme of streets and arteries being ill-proportioned in width, length, direction, and capacity.

Operation.—Congestion comes from the lack of comprehensive zoning for "use and development," for meeting occupational requirements of spacing, height, and bulk of buildings. Congestion comes from lack of zoning the density of population, that an organic plan of streets and arteries may be calculated in dimensions and correlated to the fluid—ebb and flow of population and traffic within the population-shed.

Desiderata.—An organic correlation of use, density, and dimensions will afford a normal interval of time-distance between home and work; will afford health, efficiency, and well-being. This implies fitting design to purpose in function and limitation; the planner's problem, if all remains variable, would be insoluble.

We must seek, therefore, to determine self-contained urban units of balanced functional ratios and the most efficient and desirable assembly and inter-relation of these units.

Analysis

Forces.—The forces of urban development are radial and circular and seemingly solution should be sought in approximation of the properties of

* Presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926.

† Pres., Town Planning Inst. of Canada, Ottawa, Ont., Canada.

circles—not in those of squares, as has been the traditional attempt to fit square plugs into round holes.

The principle of hexagonal planning, traffic intercepter, and orbit are submitted as affording a correlation of traffic that will obviate congestion and adjust environment to its human purpose—survival.

Forms

Blocks and Plan Pattern.—The circle is the most economic boundary (or street) to any given area, as a unit. Circular areas as units do not, however, assemble economically—they leave waste interstices. (See Fig. 3.) Allowing for flexibility and a measure of compression will, in practice eliminate the wasteful interstices and develop the hexagonal form as the nearest approximation of the subdivision virtues of circles, yet free from their structural disabilities as streets. Below ground, sewers, mains, conduits, etc., must be built in straight sections; above ground, buildings, rectangular walls, etc., are built more economically and advantageously in straight sections than in curves.

A system of subdivision demands a pattern of units susceptible of continuous and indefinite contiguity without intersticial areas wasteful of local improvements and public services.

Three Main Conditions.—To obviate congestion of traffic, and the many evils that come of it, there are three main conditions to be solved:

1.—The distribution of population in its homes, for which hexagonal planning is submitted as being most advantageous. (See Fig. 1.)

2.—Transportation between home and work, for which the intercepter affords the optimum time-distance. (See Figs. 2 and 4.)

3.—Routing the process of "work", for which consider a correlated traffic orbit. (See Figs. 4 and 5.)

The Hexagon.—The hexagon unit block and pattern seems best to meet the requirements of a basic system and general pattern; viewing the major area of cities as residential hexagons, with ample latitude for specialized differential and rectangular design to purpose of business, commerce, and industry. (See Figs. 2 and 4.)

The advantageous properties of the hexagonal residential block are:

A.—Health.—North pointed; universal access of sunshine; affords safety, rest, recreation, and amenities for child, adult, and community lift. (See Fig. 1.)

B.—Traffic.—Safety and acceleration; three-way wide angle of vision; street junction of three-collision points *versus* sixteen in rectangular limited-visibility crossings—collectively, total of eighteen *versus* sixty-four collision points. Further, the street length around the equivalent rectangular block is 1 420 ft., that is, an average collision point for every 22.19 ft., while the length around the hexagonal block is 1 284 ft., that is, an average col-

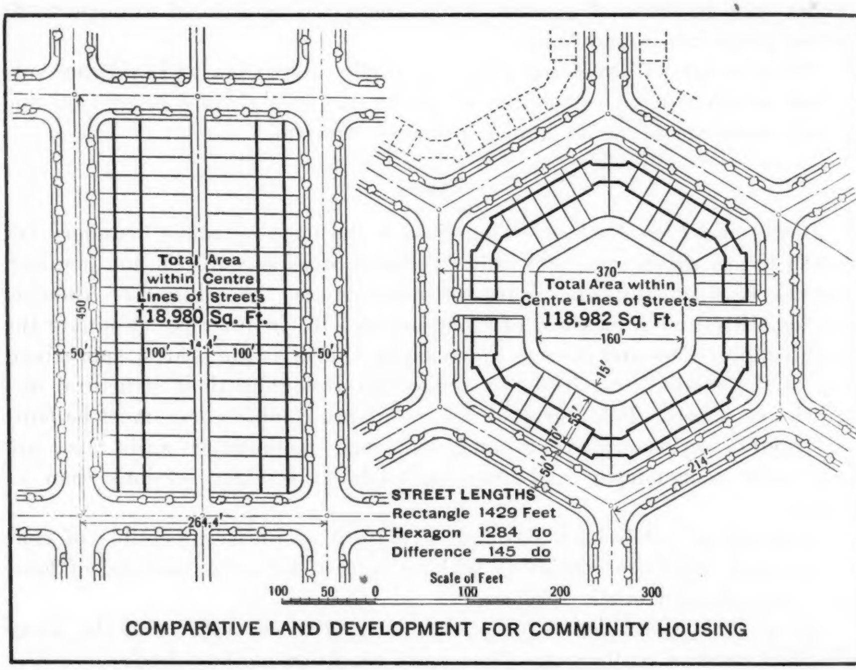


FIG. 1.—SECTION SHOWING PLAYGROUND OR GARDEN IN CENTER OF HEXAGONAL BLOCK.

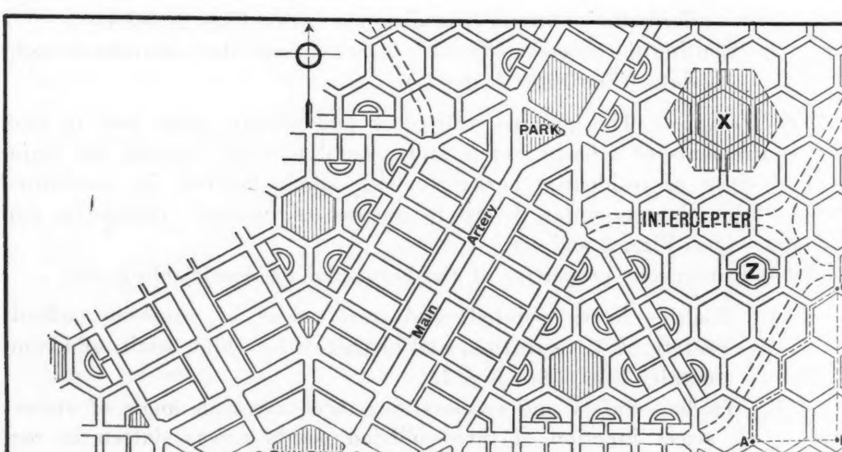


FIG. 2.—ILLUSTRATION OF RELATIVE BALANCE OF CELLS TO ARTERIES.

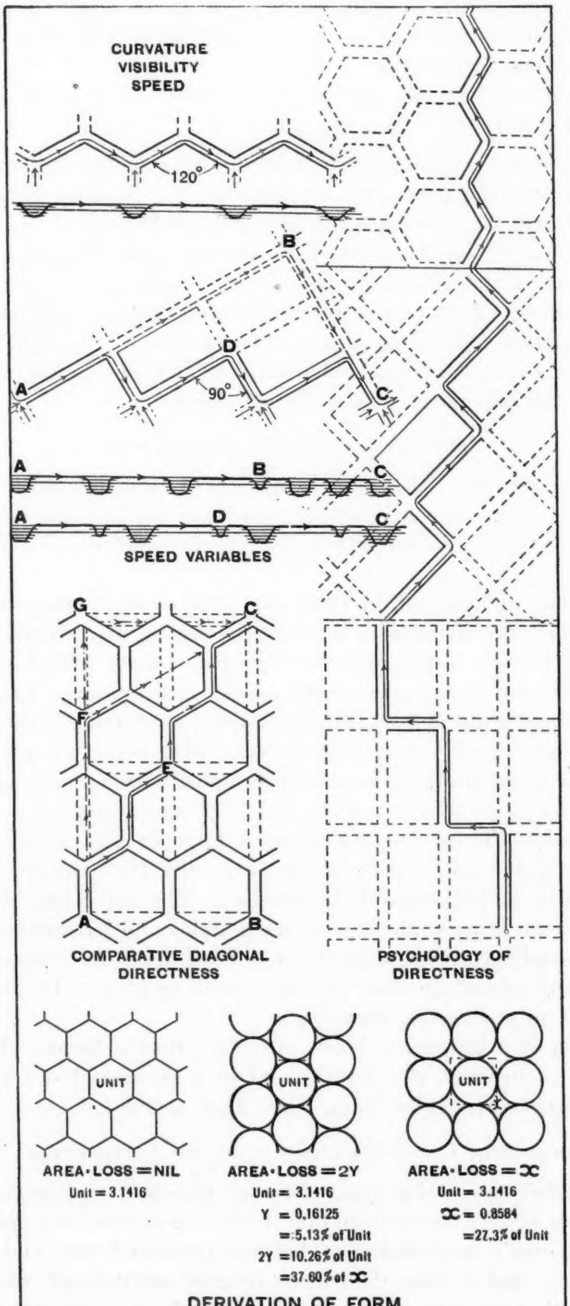

CURVATURE
VISIBILITY
SPEED
120°
B
D
A
90°
C
A
B
C
A
D
C
SPEED VARIABLES
G
C
F
E
A
B
COMPARATIVE DIAGONAL DIRECTNESS
PSYCHOLOGY OF DIRECTNESS
UNIT
UNIT
UNIT
AREA·LOSS = NIL
Unit = 3.1416
AREA·LOSS = 2Y
Unit = 3.1416
Y = 0.16125
= 5.13% of Unit
2Y = 10.26% of Unit
= 37.60% of ∞
AREA·LOSS = ∞
Unit = 3.1416
∞ = 0.8584
= 27.3% of Unit
DERIVATION OF FORM

FIG. 3.

lision point for only every 71.33 ft. of street length. Besides there is 10% less street length in the hexagonal street-planning system. (See Figs. 1 and 3.)

C.—Economy.—Fewer and shorter streets, less capital costs. (See Fig. 1.)

The psychological and actual properties of hexagonal planning may perhaps best be judged from the accompanying diagrams, particularly Fig. 3, and their indications, noting the correlated content and directness.

The Dominion Postal Authorities, at Ottawa, Ont., Canada, have kindly given consideration and thought to the effects which hexagonal planning would have on the delivery of mail, by letter carriers, on post-box collection by motor cycle, and on parcel delivery by truck. The opinion of the men in these different services is that they would find no difficulty in adopting their "bump of locality" to the hexagonal system of subdivision.

Previous Use of Hexagon Block

Mueller, in Germany, about 1909, advocated hexagon blocks, assembled so as to leave all sides as continuous straight streets; but this left triangular interstices which were economically wasteful of bounding streets and public services.*

A. R. Sennett, in England, in 1905, published a two-volume work on town planning, in which he advocated absolutely the rectangular circulatory system, even objecting strongly to the introduction of diagonals. He, however, advocated the subdivision of the rectangular blocks into lots on a hexagonal pattern, that is, three rows of lots between streets, the center row being small hexagons, off center to the street-fronting lots. The object of this disposition was to allow for equal frontage ownership, coupled with a choice of additional rear-yard area for those who wished to take such allotments.

Major Arteries.—There are examples in Europe; also in America. An hexagonal system of such arteries is shown in the 1807 Governor's Plan of Detroit,† which is the best example in America. The individual blocks in this Detroit plan are not hexagonal, they are rectangular and irregular and without any relation to advantageous orientation. Even with its limitations, however, it had many advantages over the then existing plans. Its abandonment was a great loss to progressive planning.

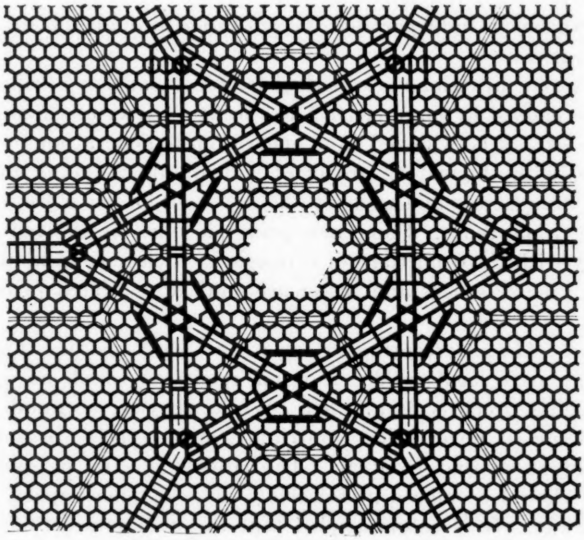
The adoption of a hexagonal block in basic patterns favors, although not necessarily nor exclusively, the development of a hexagonal system of major arteries, both surface and intercepter. (See Figs. 2 and 4.)

Collective Communications and the Intercepter

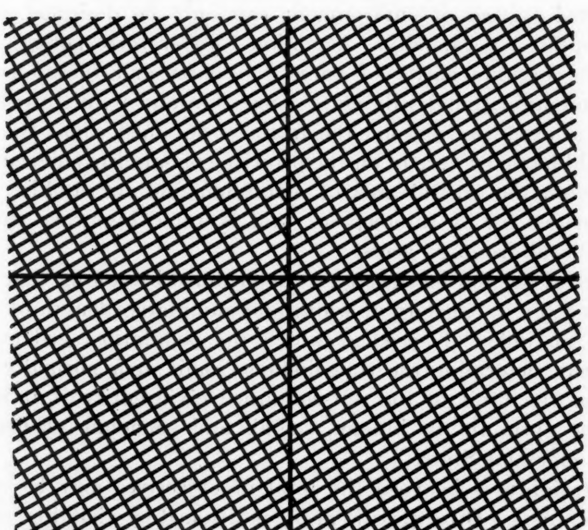
Minimum Time-Space Interval.—The requirements of communication demand of a plan system and pattern that it offer the minimum average interval in time-distance over a maximum area, between home and work and within the sum of objectives; that is, that the streets, arteries, and through transits should be so correlated that the tide of population and traffic may ebb and flow freely,

* "Town Planning," by Innigo Triggs, 1909, pp. 113–117.

† See Report, Plan Comm. of Detroit, Mich., 1924.



HEXAGONS, INTERCEPTER, AND ORBIT.



RECTANGULAR GRIDIRON AND DIAGONALS.

FIG. 4.—DIAGRAMS SHOWING COMPARATIVE CIRCULATION.

without congestion, over the greatest possible area. This should be for a normal given time-distance of about 30 min., or so much more as psychology and circumstance may demark as the turning point of amenity in free attraction, competitive areas, and inducement to spread. (See Figs. 2 and 4.) The intercepter is shown in light parallel lines running through the small hexagons by grade separations.

Present arterial provisions are inadequate for expansion because they ultimately become choked by cumulative local traffic. Arteries calculated for an initial purpose cannot be widened from generation to generation, as the process will eventuate in the logical absurdity of continually reducing the economic proportionate dimensions and area of the business lot on a thoroughfare of ever-increasing importance. Some new principle must be evolved for dealing more efficiently with the conditions of expansion which have come with modern civilization in greater speeds and vaster accumulations. That principle is the intercepter.

Intercepters.—The intercepting artery is one which carries through traffic free from cumulative local traffic; it is an artery on which no opening and traffic from facing property is allowed. While denying accumulative access along its frontage, it nevertheless contributes open spaces for sunshine and air to the windows of that frontage. The adits and exits of such an intercepting artery should be placed at relatively long distances apart, as are stations on a railway. It is recommended that the distances between the adits and exits of an intercepting artery be about ½ mile apart.

A correlated system of environment can only be designed scientifically for given purpose and capacity. Accretion (*laisser faire* policy) beyond the designed given organic entity of correlated parts and functions (ratios of forms and communications) will cause congestion and its disintegrating processes. Organic expansion beyond the initially designed capacity of the civic machine should be taken care of on the principle of satellite sub-centers. Interrelations between the central area of attraction and its satellite sub-centers should be provided for by intercepters, that is, rapid highways free from the accretions of local frontage access.

The factors of population density, as determined by zoning (use and bulk), and as provided for in capacity of width and cumulative length of streets and arteries, with an index of speed at which the population-shed can mobilize its fluid content, determine the time-distance radii of a normal civic entity. This should be the limit of the self-contained civic entity if it is to attain and retain the optimum in human, social, and economic efficiency of the individual, in city and State.

Average straightness between the total of objectives may be summed up as general directness. Directness is also conditioned by visibility accelerating speed. Further, there are factors of psychology in directness, the appearance *versus* the reality. (See Figs. 2, 3, and 4.)

Cost

Relief versus Congestion.—The question of the cost of land required for intercepters and the cost of their construction is in principle answered by com-

parison, on its merits, in given circumstances, with the cost of congestion, where lack of width and freedom develop it. And what does it cost to relieve it by widenings, subways, viaducts, etc., prohibitive as a general process?

Where the relief by widening, etc., is only local, it frequently merely defeats its own ends by inducing a diversion of traffic to itself and renewing congestion, but to a more intense degree.

Individual surface diagonals by their more outstanding directness, where not part of an organic system, are also apt to develop congestion and contingent disabilities, although they are often the lesser of two evils.

The Gridiron

The gridiron street system is a relic of primitive two-dimensional thinking, merely length and breadth. It develops the illusion that from any street intersection in any area there is a straight way to all other objective points within the area. Whereas the gridiron favors straight ways in merely two directions, it presents the long way round, two sides of a triangle instead of the diagonal, to the great majority of destinations. (See Figs. 3 and 4.)

The illusion of general directness in the gridiron has been fostered by the coincidence that city maps are conventionally made heading to the north, by which the apparent mass of streets is leading freely out and away from the observer (look at Fig. 4 according to north point); the eye seems more inclined to sweep the vertical plane and with less effort than the horizontal. When a gridiron plan is studied at an angle of 45°, cornerwise, for traffic relief, the indirection of routing arouses an almost instinctive longing for diagonal directness. (See Fig. 4.)

The thoughtless license given to population density by heights abnormal to the channels of communication awakened engineers to a third dimension, namely, volume. Very soon recognition of time will follow—time-distance in speed and safety—as the qualifying fourth, the limiting one of capacity.

A balanced correlation of forms and contents, of channels and speeds, must now be sought, the norm of entities and inter-relations that will afford freedom from congestion; for congestion is the cancer of community progress.

The Orbit

In an accompanying schematic diagram (Fig. 4) a field of hexagonal residential blocks and streets is shown, served by a twin system of surface arteries for local distribution and diffusion, circumscribing a large open central area. The mobilization of traffic throughout the population-shed is compensated and accelerated at "vortex" contacts between the local arteries and the through-traffic intercepters. (Intercepters shown in light double line, Figs. 2 and 4.)

The diagram of comparative circulation (Figs. 4 and 5) is to illustrate a theory that major arteries in converging, as they should, toward a central area, still ought to diffuse far short of a central point. There should be also a relatively large park-like inner central and pivotal area of low pressure; further, that there would be engendered by a design of this nature a constant disposition to division and deflection of incoming and outgoing movements, establishing balanced centripetal and centrifugal mass movements of traffic,

that would set up a form of gravitational attraction swaying and converting the inevitable peaking of the focal surge of traffic into rotation on a well-balanced orbit.* (See Figs. 4 and 5.)

Instinct of Survival.—The instinct of racial survival has engendered a dominant sense of community welfare justifying the view of the Hon. W. L. McKenzie that "private rights should cease when they become public wrongs"—the formula of liberty under which we should operate—all government being but synthetic organization for the maintenance of life.

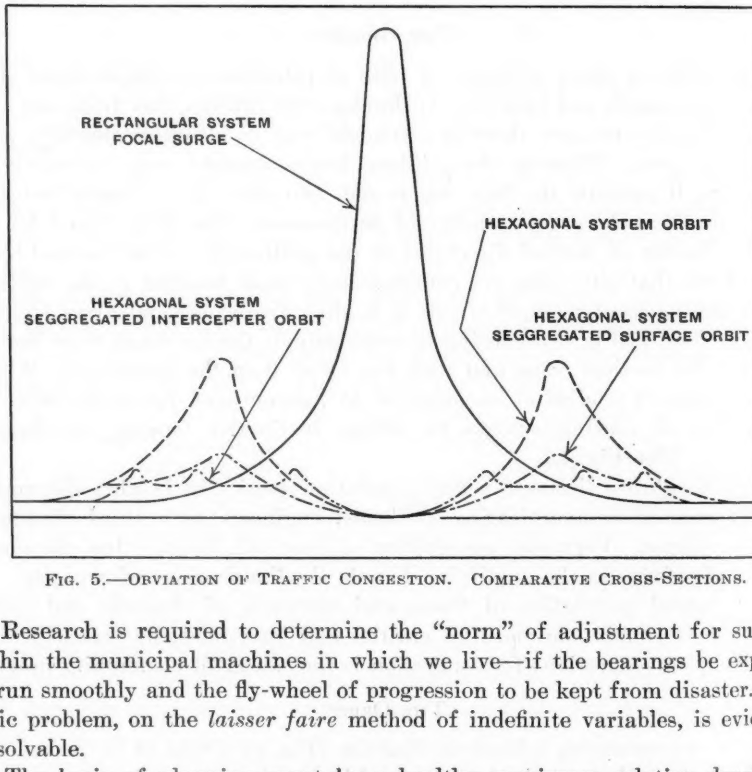


FIG. 5.—OBVIATION OF TRAFFIC CONGESTION. COMPARATIVE CROSS-SECTIONS.

Research is required to determine the "norm" of adjustment for survival within the municipal machines in which we live—if the bearings be expected to run smoothly and the fly-wheel of progression to be kept from disaster. The civic problem, on the *laisser faire* method of indefinite variables, is evidently unsolvable.

The basis of planning must be a healthy maximum relative density of occupational living conditions; and, as a principle of racial survival, the point of permissible saturation must not exceed that of social well-being. In the ultimate higher economics of Ruskin, "there is no wealth but life"—anything less is only a measure of commercial cannibalism.

"Reform delayed", said Edmund Burke, "is revolution begun". Now get your public suffused with four-dimensional consciousness—and thinking—and "we're off!"

* Compare comparative cross-sections of focal surge peak *vs.* gravitational orbit.

THE RELATION OF HIGHWAY TRANSPORTATION TO THE RAILWAY*

By Ralph Budd,† M. Am. Soc. C. E.

In the past twenty-five years the American public has increased the investment in its transportation plant from $10 500 000 000 to $50 000 000 000, and has increased its annual expenditure for transportation of property and persons from $1 500 000 000 to between $18 000 000 000 and $20 000 000 000. The change has been most rapid in the last five years, when the investment has increased from $36 000 000 000 to $50 000 000 000, and the annual transportation charge from about $12 000 000 000 to more than $18 000 000 000. This increase in the annual transportation charge in five years has been due entirely to the increased expenditure on highway travel, which has more than doubled in that time, while the charge for railway transportation has actually declined.

The public is supporting two transportation plants, in each of which is invested upward of $25 000 000 000. The property owned by the railway companies is reasonably permanent, while the equipment used on the highways is of more transient character. The annual cost of transportation furnished by the railways is about one-half that produced on the highways.

An inventory of the nation's transportation system at the beginning of 1926 would have disclosed something like the following:

Railways and Equipment.		Improved Highways and Motor Vehicles.	
Miles	251 000	Miles	495 000
Locomotives	70 000	Motor trucks....	2 500 000
Freight cars	2 440 000	Automobiles	17 430 000
Passenger cars ..	56 500	Motor buses	70 000
Rail motor cars.	500		
Total units	2 567 000	Total units	20 000 000
Investment	$25 000 000 000	Investment	$25 000 000 000
Annual cost	$6 310 000 000	Annual cost	$12 125 000 000

* Presented at the Spring Meeting, Kansas City, Mo., April 14, 1926.

† Pres., G. N. Ry., St. Paul, Minn.

Five years ago it would have been like this:

Railways and Equipment.		Improved Highways and Motor Vehicles.	
Miles	253 000	Miles	370 000
Locomotives	70 600	Motor trucks ..	1 000 000
Freight cars ...	2 400 000	Automobiles	8 220 000
Passenger cars ..	56 150	Motor buses	5 000
Rail motor cars..	50		
Total units	2 526 800	Total units	9 225 000
Investment	$22 000 000 000	Investment	$13 800 000 000
Annual cost	$6 360 000 000	Annual cost	$6 000 000 000

Twenty-five years ago the inventory would have been blank so far as modern highway transportation is concerned; railway investment would have been about $10 500 000 000, and the annual cost of railway transportation about $1 500 000 000, or only one-twelfth as much as the country's present annual transportation bill.

Influence of Transportation

Means of communication always have been a controlling factor in the life of every country. Until the Nineteenth Century, water transportation was so much cheaper and more efficient than any other, that population and industry concentrated only where it was available.

The most important single factor which influenced the character of settlement in the interior parts of America was the substitution of overland transportation by rail for that by canal, river, and highway. The railway eliminated the backwoods and caused cities to be built at what had been the frontier. It is not too much to say that the political unity of the United States was preserved largely by the railways, which alone made communication between parts of so vast a Commonwealth practicable and convenient. Without them, people in some sections of the country might have found it more advantageous to trade with foreigners than at home; and in a continent where National lines were forming, allegiance well might have followed the course of commerce.

George Washington was much concerned about the remoteness and inaccessibility of the country lying west of the Allegheny Mountains, and to overcome the physical difficulties of communicating with the Atlantic seaboard he investigated the trade routes by which the Great Lakes and the Ohio and Mississippi Rivers could be reached most easily from the East. These routes were to be developed by improving the rivers, making them navigable as far up stream as possible, and then connecting the heads of navigation on the opposite sides of the mountains by highways. He mapped routes by way of the Hudson and Mohawk Valleys, and by the passes at the head-waters of the Potomac, the James, and the Juniata Fork of the Susquehanna. Each of these routes was occupied by an important rail-

way line soon after the supremacy of the railway over other forms of transportation became recognized.

The anxiety of the Father of his Country would have been greatly relieved could he have known of the revolutionary changes in inland transportation to come within fifty years after his death, and that the railroads not only would carry the commerce between the western valleys and the Atlantic seaboard, but would be extended 2 000 miles beyond the westerly outposts of his time, crossing three major mountain ranges, knitting the political and commercial life of the nation from Atlantic to Pacific, and providing incidentally the long sought route to India over, rather than around or through, the continent.

Railways Essential

The railways to-day are as essential to the national and commercial life of the United States as ever, and anything that would jeopardize their success or efficiency should be avoided as a public menace. Other forms of transportation, however, are factors too, and it is well to consider them in their relationship to the railways and to the general transportation scheme.

In the development of transportation one form has succeeded another with astonishing rapidity, but not without a struggle; thus, we find operators of pack trains contesting with Conestoga wagon drivers, canal companies resisting railway projects, the graceful, yacht-like clipper ships yielding reluctantly to steamships, and steam railways competing with interurban electric lines for local passenger travel. Now, steam and electric lines, which had surpassed all others in the field of transportation, have encountered something that excels them both in certain particulars and under certain conditions. They find local traffic is taken from them by the most universal of all carriers, the motor car on the highway. As in former competitions between old and new means of transport, that which gives most of what the public wants will win. There must be speed, safety, dependability, comfort, convenience, and, in the case of public carriers, economy.

Automobile Industry

That most phenomenal of all industrial developments, the automobile industry, is the youngest, and now is said to be the largest, in the United States. It is barely twenty-five years old. Its importance is so great, taken as a whole, that the railways gain much more from the freight traffic it gives them than they lose from the freight and passenger business it takes away.

Like all great developments, that of motor travel has been the result of a combination of favorable circumstances. Most important were the perfection of the gasoline engine and the paved highway, which latter depended largely on good, cheap cement. Added to these is the fact that in America there is a standard of living so high that luxuries are not beyond the reach of the many. Each of these conditions is partly the cause and partly the consequence of the others. Of all the automobiles in the world 83% are in

this country, which has about 7% of the world's population. Even more are produced in the United States than are used, but it is a mistake to think that the automobile originated here, or that it always has been peculiarly American. Before the manufacture of automobiles was of any importance in the United States, they were in more or less common use in England and on the Continent, and had reached a much higher state of perfection there than here. It was not until about 1905 that the number of cars in the United States exceeded the number in Great Britain. There was comparatively little improved highway in this country then, but there were magnificent distances which afforded an opportunity for the automobile to attain its fullest capabilities. Moreover, the great individual purchasing power of the population constituted a potential demand which required only the encouragement of reasonably priced, reliable cars and better highways to burst into actuality. The volume of this demand made quantity production possible and brought the low priced car, together with a program of general highway improvement throughout the country. The almost universal ownership of the automobile which has resulted demonstrates the fact that when the public finds something it approves of and desires, its response is quick and emphatic. The new contenders for local freight and passenger traffic—the motor truck and bus—are outgrowths of the automobile.

Why the Motor Bus?

Probably the questions most commonly asked by railway men concerning the motor bus, are "What can its attraction be?" and "Is it not a fad which soon will lose its novelty and disappear?" Let us consider these questions. In many localities the bus does have some advantages over the railway train for local travel. Two of these are the greater frequency and the flexibility of its service. Compared with the railway train, the bus can give service at more frequent intervals, because each unit of service is small and may be operated cheaply in comparison with the cost of operating a train.

The ratio of cost of highway bus, to steam train, operation is about 1 to 5, which means that for the cost of one train in each direction, say, morning and evening, a bus can be run every 2 hours in each direction from 8.00 A. M. to 4.00 P. M., and this more frequent service better suits the needs of the average rural community. Owing to the extensive use of the private automobile there is scarcely enough travel even morning and evening on the average local run to justify one train, much less to justify several trains during the day; but the smaller and less expensive motor bus operating on the highway may pick up sufficient traffic to make it profitable. Besides greater frequency, there is the advantage of more convenient starting and stopping places. The motor bus is able to take on and discharge passengers at any street corner or at any house along the road. In other words, the motor bus is able to give a more flexible service than the train. People in the country can hardly use the railway for travel between neighboring stations, because, in proportion to the whole journey, the trips to and from the stations are so long. Not so with the bus. It gives continuous service all along

the highway, while the railway gives it only at points four to six miles apart. Now, the amount of this strictly local business which railways cannot handle is considerable, and may be enough to insure the success of bus transportation.

Rail motor cars are being used rather extensively in lieu of steam passenger trains. They provide a unit of more suitable size, and economize by substituting the internal combustion engine for the steam locomotive, as well as in other ways. About five hundred such cars of various types are in service, and the cost per mile for operation is about one-third that of a passenger train. They are successful, therefore, to that extent, but are subject to the inherent limitations of any vehicle operating on railroad right of way. They cannot get as much "pick-up" business as buses, which run along the highways and streets, and stop at houses, stores, offices, hotels, and any other desired place. The special field for the rail motor car is to take the place of the steam train on light traffic runs, such as branches and local and suburban districts where, for various reasons, service must be provided.

At recent hearings before the Minnesota Railroad and Warehouse Commission, Edgar Zelle, President of the Jefferson Highway Transportation Company, presented an analysis of the train and bus schedules in the territory served by his line south of St. Paul and Minneapolis. The substance of what he said in respect to one community is quoted here:

"Owatonna is a town 77 miles from Minneapolis on two lines of railway. Seven daily trains give direct service to and from Minneapolis, but the schedules are such that service is concentrated morning and evening, without any trains during long intervening periods. For example, of the seven north-bound trains, three leave within one hour and seventeen minutes of each other, with a fourth trailing just an hour later, all four of these trains leaving before 7:30 A. M. After this there is no more morning train service, and only three more trains left for the balance of the day. One of these, a limited, leaves at 1:10 P. M., and then the other two locals keep each other company, both of them leaving around 4:00 P. M. within thirty-eight minutes of each other. Thus, six-sevenths, or 86%, of the north-bound rail passenger service at Owatonna is used to give service at but two periods of the day.

"South-bound service shows another abundance of rail service at two particular periods. A train arrives at 10:23 A. M. with another close behind at 11:30 A. M. Then everything is quiet until 5:20 P. M., when the first train arrives, with another at 6:43 P. M., followed immediately at 6:59 P. M., with still another. On south-bound service the railroads thus concentrate five-sevenths of their passenger service at two periods of the day.

"The Jefferson Company, on the other hand, because it uses a type of equipment that can economically be distributed for local passenger service, gives Owatonna service from the north every two hours from eleven o'clock in the morning to eleven at night.

"That the public appreciates a frequency of service that is spread over the day at regular intervals is illustrated by the traffic records of the Jefferson Company. Over the twelve-month period ending August 31, 1925, there is a surprising uniformity of patronage, ranging from 27 to 38 passengers handled daily on each of these two hour scheduled south-bound runs. The north-bound records show a similar uniformity beginning with the first through run out of Mason City, leaving at 7:15 A.M., which carried 13 323 pas-

sengers, to the 5:15 P. M. run, which carried 15 123 over a period of twelve months, ranging from 31 to 41 passengers handled daily on each of these north-bound runs.

"The same uniformity is illustrated in the 19 831 passengers who used the outbound service at Owatonna. The pleasant month of June, with 1 342 outbound passengers, was the lightest month, while the cold month of January was the heaviest month, when 1 959 outbound passengers were taken out of Owatonna, averaging 45 per day in June to 63 per day in January.

"Owatonna, credited in the last census with a population of 7 252, furnished the Jefferson buses with a total of 37 928 in and out bound passengers in the twelve month period.

"This two-hour bus service is not only patronized at small stations where the railroads restrict their service, but also at any point between stations. The cross-roads or any point on the highway is the stopping place of the bus."

While inapplicable to the territory adjacent to the largest cities or to sparsely settled regions, the condition described by Mr. Zelle is fairly typical of a great part of the United States.

The radius of travel of an individual multiplies many times when he becomes the owner of an automobile. His sense of independence and freedom, and his ability to give himself and his family enjoyment not otherwise obtainable, are sufficient reasons for sacrifices, if necessary, in other directions in order to have a car. For short-distance travel the most ideal way yet devised is by the private automobile. This is an important truth, because it accounts for most of the development in motor-bus transportation and most of the railways' loss of passenger traffic. For those who do not have their own automobiles, or having them, prefer occasionally not to drive, the motor bus affords a substitute.

The congestion of city streets has become a serious problem for the automobile user. In all cities, during the busiest hours of the day, much of the advantage of the automobile is lost for lack of parking space on the streets. This problem is having attention, and doubtless, to some extent, it will be solved by providing convenient places for parking cars near business centers. The cost of such parking, however, will influence some private car users to avoid the congested centers. In very large cities the bulk of commutation travel probably can be handled only by railway trains, subways, and elevated lines, but there seem to be many cities where the street congestion is not too great for motor buses, yet is too great for private cars to operate conveniently, comfortably, and economically. In such places the motor bus has positive advantages.

The Automobile and Local Travel

In connection with these questions of frequency and flexibility of service, which are the main advantages of local highway over local railway passenger service, consider whether the railways really lost their business to motor buses or to private automobiles. Statements submitted to the Minnesota Railroad and Warehouse Commission recently indicate that the railways in Minnesota had lost a substantial part of their local passenger traffic before motor buses began operating to any extent, and that the number of

automobiles continued to increase as the number of passengers carried by railways declined; also that at stations where motor buses have been operating for some time, the loss of passenger business has not been materially greater than at stations where they never have operated.

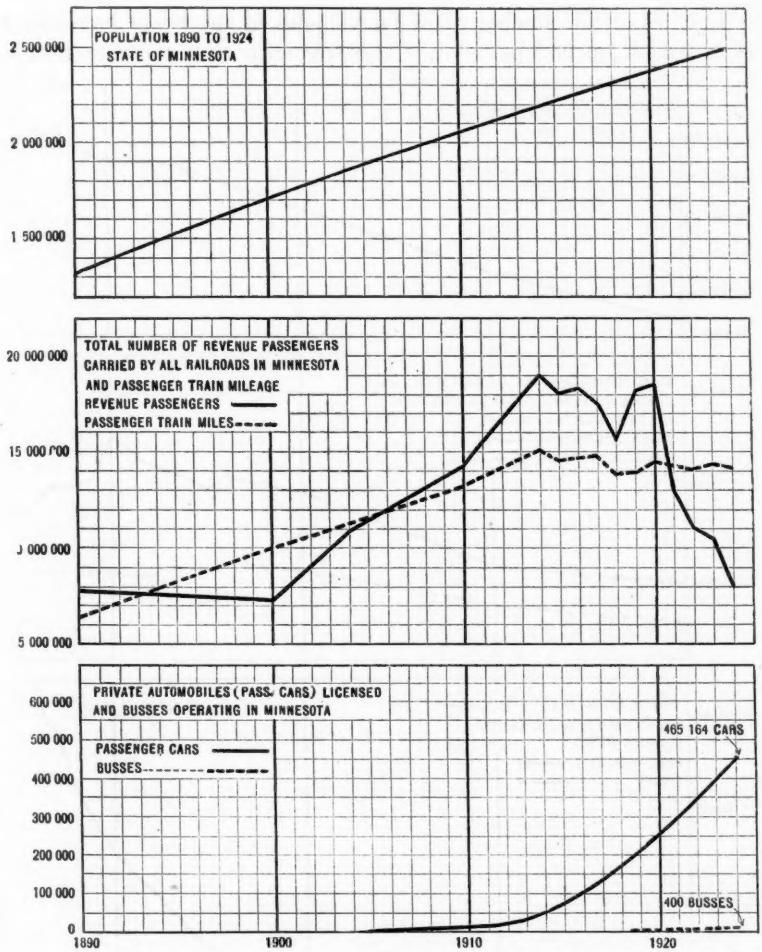


FIG. 1.

Fig. 1 shows the number of passengers handled by the railways in Minnesota since 1890, passenger-train miles, automobiles, and buses in Minnesota, and population; Fig. 2 gives similar information for the United States; and Table 1 shows the ticket sales at twenty-six railway stations in Minnesota. At fifteen stations, where there was no bus competition, the decrease in passenger tickets sold in 1924 as compared with 1920, was 49 to 76%, with an average of 64.6 per cent. At eleven others, where there was bus competition, the decrease was 55 to 74%, with an average of 63.7 per cent. The

total number of tickets sold at the twenty-six stations in 1920 was 488 649, and in 1924 was 175 706, a decrease of 312 943, or 64 per cent. During the five years, 1919 to 1924, the total number of passengers handled by the railways in Minnesota decreased from 18 274 516 to 7 905 378, or 56.7%; while passenger-train miles on these railways increased from 14 052 547 to 14 223-456, or 1.2%; and the number of motor vehicles in the State increased from 259 741 to 503 437, or 93.8 per cent. Compared with 1919, the year 1921 shows

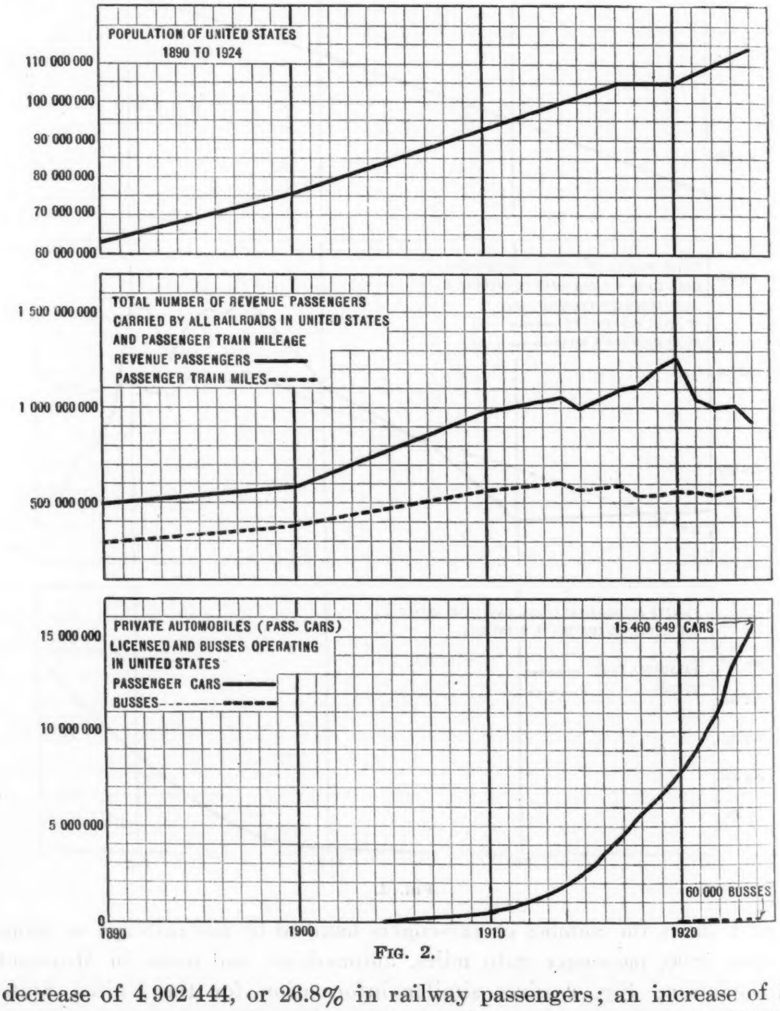


FIG. 2.

a decrease of 4 902 444, or 26.8% in railway passengers; an increase of 535-584, or 3.8% in passenger-train miles; and an increase of about 60 000, or 25%, in automobiles, while buses had not yet become a factor. These and other data point to the conclusion that the private automobile has had a great deal more to do with the loss of railway passenger business in Minnesota than the motor bus. They also suggest, and railway statistics support, the

suspicion that before bus operation began, the local passenger traffic of the railways in Minnesota had decreased to a point where much of it was being done at a loss, largely because passenger-train miles had not been correspondingly reduced. Manifestly, the taking off of passenger trains in such instances is justified. Every train that is removed, however, serves to increase the advantage of the competitor on the highways; but if the business disappeared while the railway service was maintained, there is hardly reason for continuing such trains. Whatever may be the various reasons, local travel, to a very large extent, has left the railway train for the automobile and bus. This patronage of buses seems to establish beyond any doubt that they will continue, and probably will increase in number. From the foregoing it seems clear that the railways must recognize that public necessity and convenience require the development of transportation on the high-

TABLE 1.—RECORD OF TICKETS SOLD IN 1920 AND 1924, IN MINNESOTA.

Station.	Number of Tickets Sold.		Decrease.	Percentage.
	1920.	1924.		
At Fifteen Stations Where There Was No Bus Competition.				
Benson	35 371	11 557	23 814	67
Browns Valley	5 882	1 773	4 109	70
Clara City	7 665	2 420	5 245	68
Cottonwood	7 371	2 393	4 978	68
Granite Falls	16 680	5 583	11 097	67
Hallock	10 569	5 345	5 224	49
Hanley Falls	10 925	3 011	7 914	72
Herman	8 196	3 099	5 097	62
Monticello	12 924	4 600	8 324	64
Ruthton	5 790	1 722	4 068	70
Warren	16 844	7 739	9 105	54
Ortonville	2 743	1 005	1 738	63
Odessa	272	66	206	76
Appleton	10 562	3 268	7 294	69
Milan	312	115	197	63
Total	152 106	53 696	98 410	64.6
At Eleven Stations Where There Was Bus Competition.				
Alexandria	37 398	15 429	21 969	59
Delano	11 376	2 909	8 467	74
Evansville	10 491	3 153	7 338	70
Fergus Falls	59 422	26 868	32 554	55
Jasper	12 624	4 460	8 164	65
Litchfield	38 094	10 310	27 784	73
Marshall	27 312	10 658	16 654	61
Osakis	14 790	4 184	10 606	72
Park Rapids	13 459	5 612	7 847	58
Sauk Center	35 244	10 519	24 725	70
Willmar	76 333	27 908	48 425	63
Total	336 543	122 010	214 533	63.7
Grand total	488 649	175 706	312 943	64.0

ways; that they should not attempt by arbitrary means to eliminate motor-vehicle competition, and should only insist that such competition be subject to proper public control; and, further, that they should seriously consider whether or not this new form of transportation, from the public as well as their own point of view, cannot be more advantageously conducted under railway direction than otherwise.

Regulation

The rapid development of the common carrier motor vehicle, especially as embodied in the bus, has resulted in the enactment of regulatory measures by thirty-seven States. Similar measures are under consideration by legislative bodies of other States. The important and main provisions of these regulatory acts grant to the State the power to determine whether or not common carrier motor vehicles should be permitted to operate. If the State regulatory body is of the opinion that the public interests demand the operation of such motor-vehicle service, it issues a certificate therefor which is commonly called a "certificate of public convenience and necessity." No operation of common carrier motor vehicles can be carried on in these States without obtaining such a certificate. These laws,

1.—Require an adequate bond to protect the traveling public.
2.—Require State control of rates charged.
3.—Require State control of schedules of operation.
4.—Prohibit discrimination between individuals and communities.
5.—Require safety of operation with reference to the type of vehicle, and in other details affecting the safety of the traveling public.

These acts grant to the State regulatory boards the same control in general which these boards exercise over railway carriers. The regulatory board is required to give consideration to the effect the proposed service may have on other carriers, whether those carriers be railways or other motor-vehicle carriers.

These regulatory acts are wholesome and were necessary. Without such regulation the public had no protection from so-called "fly-by-night" operators, who had no capital and who were unable to furnish adequate service. These irresponsible operators would come and go as the seasonal business might permit. That made it impossible for the legitimate operator to make reasonable profits and maintain adequate service. Sound regulation is building up in the several States a fixed and dependable service, and one which, in co-ordination with the railway service, gives to the public the best conceivable local transportation. The rates which the bus companies are permitted to charge in general bear a fair relation to the railway rates, and are generally only a fraction of a cent per passenger mile less than railway fares.*

The States have no power to forbid the operation of interstate carriers and have very little regulatory power over them. The Supreme Court of the United States has held that a State law which prohibits common carriers for hire from using the highways by motor vehicles between fixed termini or over

* A tabulation of the State Laws in force as of January 1, 1926, has been made by the National Automobile Chamber of Commerce.

regular routes without having obtained from the Director of Public Works a certificate declaring that public convenience and necessity require such operation, is primarily not a regulation to secure safety on highways, or to conserve them, but a prohibition of competition; and, as applied to one desirous of using the highways as a common carrier of passengers and express purely in interstate commerce, is a violation of the commerce clause, besides defeating the purposes expressed in Acts of Congress giving Federal aid for the construction of interstate highways. Prior to this decision, many of the States were regulating interstate carriers.

Truck and bus operators engaged in interstate commerce recognize that Federal regulation sooner or later is inevitable. The Federal Government, however, in regulating the interstate motor carriers, should leave that regulation, so far as the Constitution will permit, to the Commissions of the interested States. About 75% of the railway business is interstate, and, therefore, the regulatory power of the railways is properly vested in the Federal Government. The truck and bus business, by its nature, always will remain largely a local problem. Perhaps as much as 90% of the truck and bus transportation of the country always will be intrastate. Those vehicles travel highways which were built by the State, and are policed and maintained by the State. It is, therefore, most proper that the power that regulates be delegated to the several States as far as practicable.

Congress has under consideration a bill regulating interstate motor-vehicle transportation. Its main features are as follows:

No common carrier truck or bus can be operated in interstate commerce without obtaining a certificate of public convenience and necessity therefor.

This certificate is to be obtained from a Joint Board composed of representatives from the several States in which the applicant proposes to operate. An appeal from the decision of the Joint Board so created, lies to the Interstate Commerce Commission.

The Joint Board has complete power to regulate the service and rates and safety of operation of such motor vehicles. The bill as drawn is intended not to hinder or hamper the development of common carrier transportation upon the highways, but to protect the legitimate operator thereon. It recognizes as a fundamental principle that common carrier transportation service must be in the hands of a responsible operator and that he should be protected from the irresponsible and casual operator.

The necessity of Federal regulation in the large States is not acute for the reason that nearly all such carriers are engaged in intrastate transportation and are subject to the regulation of the State. However, where these transportation companies are operating in the smaller States and it is possible to cross State lines in a normal day's operation, the necessity of Federal regulation is apparent. The public cannot be adequately protected unless the Federal Government enacts legislation that will require the operating company to furnish security for damage to persons or property, and unless the service and rates of such operators are fixed and determined through public authority.

The proponents of the bill have agreed that it should provide that operating companies, which were in operation on March 1, 1925, should, as a

matter of course, without further proof of public convenience and necessity, receive a certificate.

Taxation

Every user of facilities furnished by the State should pay reasonable compensation for the use thereof, especially when such facilities are used for private gain. Common carrier motor vehicles should pay a fair and reasonable tax for the use of the highways, but regulation should not be attempted through taxation; that is, taxes should not influence the granting of permission to operate, and they should not be burdensome to the point of preventing low fares.

A highway is constructed for the benefit of society as a whole. Society, in the aggregate, benefits whether or not each individual may or may not use the highway. It has never been the policy of a State to charge the entire cost of upkeep of the highway to the users thereof. The highways are used by private individuals in the transaction of their private business for profit. They are used also by those who travel for pleasure, and they are used by common carrier transportation companies. Those individuals who choose not to drive their own cars, but to ride in common carrier motor vehicles, should not be asked to bear an unfair share of the burden of upkeep of the highways, nor should they be deprived of the advantage of cheap transport as inevitably must happen if public motor vehicles are taxed unduly, resulting in higher fares.

There are about as many different methods of taxing common carrier motor vehicles as there are States in the Union. Some States tax these vehicles solely on the basis of value or cost, some on a percentage of the gross revenue received, some on weight plus a fraction of a cent per passenger mile, and some on the seating capacity of the vehicle, or on the horse-power, or so much per hundred pounds of weight, or a combination of all. In addition to these forms of taxation, nearly every State has a per gallon gasoline tax. This tax amounts to about one-third of a cent per mile for the common carrier motor vehicle. Speaking generally, the taxes paid by buses are fair and just. In Minnesota, the tax is based on the value of the vehicle. The parlor-car type of bus in common use in Minnesota costs from \$10 000 to \$12 000. The annual tax paid on each vehicle is 10% on cost for the first year, decreasing 10% per annum to a minimum of \$350. The average for the modern bus is about \$750. The average annual tax paid on a Ford car is \$12. It thus appears that the annual tax on a bus is about sixty times that on a Ford car. In addition, a tax of \$0.02 per gal. is paid upon each gallon of gasoline used. The writer is advised that the tax paid by the bus companies in Minnesota is between 6 and 7% of the gross earnings of the operating companies. The tax on gross railway revenue in that State is 5 per cent.

The annual tax on a standard parlor car bus of a seating capacity of 30 passengers, weighing approximately 10 000 lb., and costing \$10 000, would vary in the different States from \$150 per bus per year to \$1 000 per bus per year.

Commercial highway users themselves have taken an active part in forming an enlightened public opinion on the questions of regulation and taxation. In January, 1926, the Motor Vehicle Conference Committee published, among other articles, the following:

1.—Sound and Equitable Principles for Intra-State Regulation,

2.—Recommended Restrictions on Motor Vehicle Sizes, Weights, and Speeds,

3.—Sound and Equitable Principles to Control Special Taxation for Motor Vehicles.

These are attached as Appendices A, B, and C, respectively.*

One may not agree with the details of these recommendations, but it must be conceded that the free and unrestricted use of the highways by commercial vehicles in competition with the railways is a thing of the past, and that the operators themselves recognize the desirability of having their use of the highways controlled by appropriate laws rationally administered.

Whether a railway company itself should own and manage buses may depend on its willingness or unwillingness to take on additional obligations and responsibilities; but if no prejudice exists against bus operation, the deciding question probably will be whether, by such control, wasteful duplication can be eliminated and the service improved. There have been instances where, by co-ordinating the schedules, bus service has supplemented train service, to the end that for a lesser total expenditure a more complete and satisfactory service has been rendered. Each case is one for individual consideration. In many places throughout the United States electric lines have abandoned all or part of their tracks, and substituted bus service. In other cases, notably in New England, steam roads have substituted buses and trucks for branch lines.

The National Automobile Chamber of Commerce has compiled a census of bus operation as of January 1, 1926. Of 28 145 common carrier buses reported, 5 462, or 19.4%, were owned by steam or electric railways. The number of non-common carrier buses reported was 29 605. They are used by hotels, industries, schools, and for sight-seeing and depot transfer. The geographical distribution of the buses reported is general. Of common carriers, the largest number in any State was 2 672 in New York, while Wyoming reported only 58. Ohio, with 2 454, had the most non-common carriers, while 11 for Rhode Island was the least reported.

Cost of Bus Operation

The question of cost of operating buses is vital for the future of that form of transportation, but reliable records have not been kept long enough to establish what might be called normal costs for certain routes or localities, as is the case with transportation costs on the different divisions of railway systems.

* Pamphlets containing digest of State laws governing motor vehicles may be obtained without charge from the National Automobile Chamber of Commerce, 366 Madison Avenue, New York, N. Y.

An article entitled "What Does it Cost to Operate Buses and Trucks?" has been published,* in which detailed estimates of the cost of operating a city type bus in New York are given. These costs vary from 23.6 to 30.6 cents per bus-mile, depending on whether the bus averages 200 or 100 miles per day. The details of these estimates have been compared with cost data, with which the writer is familiar, and he believes they are as reliable as can be made at this time, taking into account the fact that local conditions will determine several of the items in any such estimate. The cost of bus operation, however, should be at least 15% less than it is, and unless it is reduced that much, the business will not grow to its full possibilities.

The bus has come a long way from its origin in the motor truck, but it is not perfected yet. Lists of the different makes and types of motor-bus chassis designed exclusively for passenger transportation have also been published.† In this list are shown 96 types put out by 46 manufacturers. Standardization should result in substantial reduction in first cost, and the lessening of obsolescence would reduce the amortization or depreciation charges. The items of oil and gasoline and tires, of course, automatically would decrease if a car weighing 9 000 lb. could be substituted for one weighing 12 000 lb. Cost of insurance also will be less as the business is stabilized. With lower costs, rates can be reduced and travel increased. Many people living along bus routes would take buses to town instead of driving their own cars, especially if the fare was low enough. Here is an opportunity to render a service to rural communities, and it is a logical development in rural transportation for the bus to take the place of the private car on many occasions.

The Motor Truck

There are about 2 500 000 motor trucks in the United States. About 95% of them are non-common carriers, and are not subject to regulation as to rates or service. They are the successors of the horse-drawn warehouse, transfer, and delivery vehicles, and of the farm wagon, but the motor has given them radii of operation many times those of their predecessors. In freight, as in passenger, business the railway is supreme in the long-distance field. It is also supreme in the handling of the great volume of bulk commodities, such as coal, ore, and grain. Indeed, there is nothing in the records of truck transportation to indicate that trucks are or can be contenders for any railway freight, except where the convenience of direct door-to-door delivery, together with the saving of terminal trucking and handling, outweigh the extra ton-mile cost of moving freight by truck on the highway over the ton-mile cost of railway line haul.

The principles governing the regulation and taxation of commercial freight carriers on highways are similar to those governing buses, but the handling of freight is so different from the handling of passengers that the truck bears a relationship to railroad freight service different from that of the bus to railroad passenger service. Freight shippers are interested solely in depend-

* *The Railway Age*, March 27, 1926.

† *The Commercial Car Journal*, March 15, 1926.

able, prompt, and cheap transport, whether the shipment be over a long or a short distance. The charge for freight service is important, but the question of economy does not enter into the vast bulk of local passenger travel which moves by private automobile. The flexibility and elasticity of truck operation, that is, its ability to make door-to-door delivery and to give radial service to both rural and urban communities, gives it a large field of activity. The elimination of one or two handlings and the consequent saving in time amounts to more than the excess cost of road haul by truck over that by rail up to some undetermined distance beyond the terminal. What that distance is no one knows. So many variables enter into the problem, such as the freight available on a given route, the character of the commodities, the relative importance of direct delivery to store doors, the extent to which return loading is obtainable, climatic conditions, the condition of the highways, etc., that only actual experience can determine how far beyond the city in each instance the truck can take the place of the box car. The horse-drawn truck excels the motor truck only for such freight as involves short movement and long delay in loading and unloading; the motor truck similarly excels the railway only where the distance involved is short enough so that the saving and convenience in terminal cost and handling offset the higher cost of transit by highway over railway, and only for comparatively small units of freight.

Common carrier trucks should not be permitted to operate in competition with railways except where there is a real public convenience or necessity. The convenience of a few in obtaining a quick delivery of property should not be controlling. It is most important that regulatory bodies, before granting a certificate for the operation of trucks, should carefully analyze what effect that operation will have on the essential rail carrier. The public cannot maintain two freight-transportation agencies without paying for both; and unless each performs a service which the other cannot do economically and efficiently, both should not be supported.

Railways are using trucks to assemble freight in cities in lieu of switch engines, and in some cases operate lines of trucks in lieu of local freight trains. Especially in large and congested terminals the use of trucks, whether by the railways or by others, is economical because local freight trains, due to the light tonnage, station work, and heavy switching cost incident thereto, may be and often are unprofitable. Unlike the case of passenger traffic, freight train mileage may be reduced approximately in proportion when freight traffic declines.

Summary

The situation may be summarized as follows: The superiority of the railway for long-distance and bulk freight and passenger traffic is well established; motor-truck and bus competition is not a factor in those fields. Commercial users of highways should be subjected to reasonable regulation and taxation. Existing and proposed State and Federal legislation, generally speaking, will provide for this. The extensive ownership of automobiles and

the large mileage of improved highways have resulted in the loss of most of the local passenger travel of the railways, except in the vicinity of the largest cities. The removal of local trains has left many communities with comparatively little railway passenger service. The small amount of local travel in many such instances does not warrant more railway service, but does warrant the operation of buses at comparatively frequent intervals. The station to station, and even shorter travel, which would not go by rail, makes up a considerable portion of the bus traffic. The bus business promises to increase if better service can be given and if the cost of operation can be reduced. The additional business which buses may expect will come largely from automobile users rather than from rail patrons. Buses may serve to supplement railway transportation more effectively in some localities, if managed by railways, than if operated independently. Street railway and interurban electric lines are making extensive use of buses. The field of the private carrier motor truck is wide, taking the place, as it does, of the horse-drawn vehicle both in the city and in the country. The common carrier truck has a much narrower field, because of the competition with the private truck on the one hand, and the common carrier railway on the other hand, the latter unquestionably being more economical for any but comparatively small lots and comparatively short haul. Motor trucks relieve railway terminal congestion by moving freight direct from door of consignor to door of consignee. In many cases, this does not represent a loss to the railway because at large centers, where trucks are most used, the terminal costs may absorb the profit of rail haul on local freight.

In considering public convenience and necessity—the *sine qua non* for any permit to operate a commercial vehicle on public streets or highways—due regard should be had for the existing modes and means of transport. When essential carriers are able to give service that is measurably similar to that proposed, or when the success or efficiency of the existing essential carrier would be seriously impaired without definite and distinct improvement in service to the public, then public necessity does not warrant the new facility, and it is in the true public interest to deny the application. The public must support whatever transportation agencies are maintained and should not undertake two where one will suffice. On the other hand, where such additional facility is in the public interest and, therefore, is permitted, it should not be hampered by undue restriction or unfair taxation, but should be encouraged to operate as efficiently and cheaply as possible. Under the accepted plan of providing public transportation in this country, the service is rendered at cost; including in cost, however, a fair return on the value of the property used for transportation purposes. So long as this principle obtains, and for practical purposes that is so long as transportation is furnished by private rather than by Government agencies, it is in the true public interest to avoid unnecessary duplication of capital, and in every other reasonable way to help both old and new carriers keep down the cost of producing transportation. Cheap transportation of the highest quality is the key to much of the past and possibly more of the future prosperity.

APPENDIX A

Sound and Equitable Principles for Intrastate Regulation
(Recommended by the Motor Vehicle Conference Committee)

1.—Control over intrastate transportation of persons and property for hire, over regular routes or between fixed points, if adopted, should be exclusively in the hands of some agency of the State. No power, whatever, in the premises should be vested in the governing bodies of any political subdivision of the State.

2.—Such State control over motor vehicle common carriers should be placed in existing commissions, such as the Public Utility Commissions or other appropriate State regulatory bodies, of the various States.

3.—As a pre-requisite to the operation of the motor vehicle common carrier, the owner thereof should be obliged:

(*a*) To receive a Certificate of Public Convenience and Necessity, provided that lines in *bona fide* operation on the first calendar day of the legislative session at which the law is passed shall be presumed to be necessary to public convenience and necessity and such lines in the absence of evidence overcoming such presumption, shall receive a certificate for routes established by them.

(*b*) To take out liability insurance adequate to indemnify injuries to persons or damage to property resulting from negligent operation.

4.—The State regulatory bodies having control over motor vehicle common carriers should be vested with the powers they exercise in controlling other forms of public utilities.

5.—Taxes on motor vehicle common carriers should consist of:

(*a*) Those imposed in the particular State upon motor vehicles, the proceeds from such taxes being first applied to the maintenance, and any surplus thereof to all other costs, of highways of general motor use.

(*b*) Proper and equitable taxes in exchange for franchise rights, provided that if such taxes are adopted, an amount equivalent to those paid under (*a*) should be deducted.

6.—Legislation should be enacted enabling steam railroads, trolleys, shipping companies, and other public utilities to acquire, own, and operate motor vehicles in conjunction with the regular lines of business.

APPENDIX B

Recommended Restrictions on Motor Vehicle Sizes, Weights, and Speeds
(Recommended by the Motor Vehicle Conference Committee)

Size Restrictions

1.—Width, including load, 96 in. (Traction engines 108 in.)
2.—Height, including load, 12 ft. 6 in.

3.—Length, including load:

(*a*) Single vehicle, 30 ft.

(*b*) Combination of vehicles, 85 ft.

NOTE.—From the foregoing it is apparent that in order to admit of the safe passage of two vehicles each of which with its load is 96 in. wide, a highway at least 20 ft. in width is desirable.

WEIGHT RESTRICTIONS

1.—Single vehicular unit of four wheels or less (tractors and semi-trailers to be regarded as separate units), 28 000 lb.

2.—Any one axle of the vehicle or any additional axles of semi-trailers or trailers, 22 400 lb.

3.—Per inch width of tire measured between flanges of the rim in case of solid rubber tires:

Size of Tire	Load per inch (Maximum)
3 in.	400 lb.
$3\frac{1}{2}$ "	400 "
4 "	500 "
5 "	600 "
6 "	700 "
7 "	750 "
8 "	800 "
10 "	800 "
12 "	800 "
14 "	800 "

4.—Minimum thickness of rubber for solid rubber tires:

3; $3\frac{1}{2}$; 4; 5-in. tires	$\frac{7}{8}$ in.
6; 7; 8-in. tires	1 "
10; 12; 14-in. tires	$1\frac{1}{8}$ "

SPEED RESTRICTIONS

In the matter of speed restrictions no motor vehicle should be operated upon a public highway at a rate of speed greater than is reasonable and proper, having regard to the traffic and use of the highway, or so as to endanger the life or limb of any person or the safety of any property, and should not in any event, while upon an urban street, run at a rate of speed greater than 15 miles per hour; upon a suburban street at a rate of speed greater than 20 miles per hour; or upon any other street or highway at a speed greater than 30 miles per hour.

NOTE.—The laws of many States prescribe for the three types of thoroughfares indicated a graduated schedule of speed limits based on the kind of tire equipment of the vehicle and its gross weight. Such elaborate and detailed schedules, however, are very difficult to enforce.

SPECIAL PERMITS TO RAISE RESTRICTIONS

There are, of course, times when it is imperative on certain highways or portions thereof that the movement of vehicles bigger and heavier than those allowed by law be permitted.

To meet such situations—which should be the rare exception rather than the rule—the State, county, or municipality exercising jurisdiction over roads and bridges should be empowered under definite limitations to grant written permits for the movement of restricted vehicles to meet emergency conditions.

Special Permits to Lower Restrictions

To deal with bad frost or other similar conditions where it is essential to lower the weight or speed restrictions ordinarily enforced, the power of the State, as centralized in its highway departments or the county or local highway authorities, after consultation with and permission from the State highway department, should have power to reduce the weight or speed restrictions to points deemed essential to the preservation of highways or the safeguarding of travel.

In all such cases, however, there should be public hearings on the subject; due notice of the reduced restrictions should be given to the traveling public and the highways or portions thereof affected should be properly posted.

From the nature of the case, size restrictions on the vehicle, of course, can never be reduced.

Local Powers

Except as indicated, the subordinate political sub-divisions of the State, such as counties, cities, towns, boroughs, townships, etc., should have absolutely no power to prescribe size, weight, or speed restrictions at variance with those allowed for the State as a whole. The need for such limitations on local governing bodies is too obvious to require discussion.

APPENDIX C

Sound and Equitable Principles to Control Special Taxation for Motor Vehicles

(Recommended by the Motor Vehicle Conference Committee)

1.—The State should be the sole special taxing agency—Federal, County, and Municipal Governments should be excluded from the field.

2.—The motor vehicle tax should be simple in form and distributed in equitable and just proportion between the different types of motor vehicles.

3.—No highway should be improved by expenditure of public funds in excess of its earning capacity. The return to the public in the form of economic transportation is the sole measure of the justification for the degree of improvement.

4.—All money raised by such special taxes should be placed in the State Motor Vehicle Highway Fund and to secure the best results should be expended under the direction of the State Highway Department.

5.—The cost of building and maintaining adequate systems of highways should be distributed in an equitable relation to the benefits derived. These may be summarized as follows:

(*a*) Benefits to society in general, such as influence on education, recreation, health, fire prevention, police protection, the National defense, the postal service, living and distribution costs.

(*b*) Benefits to definite groups, such as agriculture, manufacture, labor, railroads, mining, forestry, and waterways.

(*c*) Benefits to property served.

(*d*) Benefits to the road user.

6.—For the purpose of apportioning costs in relation to benefits received, all highways may be divided into two classes: First, those used by the gen-

eral motoring public; and, second, those which perform a purely local service function.

7.—Special motor vehicle taxes should be levied and used only for the improvement and maintenance of highways used by the general public, *i. e.*, for general highway traffic flow lines.

8.—The wide variance in valuations, tax burdens, number of motor vehicles in use, and the status of highway development in the several States prevent the adoption of any fixed formula as to the proportion of the total cost of highways of general use which should be paid for from motor vehicle funds. Generally speaking, however, these principles may be set forth:

(*a*) In States where the income from motor vehicles is insufficient to meet all of the maintenance costs of highways of general motor use without undue burden to the individual motorist, such funds should be applied first to the maintenance of interstate and State highway systems.

(*b*) In States where the income from motor vehicles is sufficient to meet all maintenance costs of highways of general motor use without undue burden to the individual motorist, any surplus should be used for this class of highway reconstruction and administration costs.

(*c*) In States where the number of motor vehicles will bring in large sums in excess of maintenance without placing undue burdens upon the individual motorist, such surplus should be used to defray all the costs of maintenance and a substantial share of all of the other costs of highways of general motor use.

(*d*) In those States where the motor vehicle income is more than sufficient to meet maintenance costs of highways of general motor use without undue burden to the individual motorist, it may be found advisable to use such surplus for the purpose of defraying all or part of the costs of bond issues to expedite construction of economically desirable motor highways.

9.—Roads of a purely local interest, serving only local needs, should be financed out of local revenues obtained from local general taxes. Special assessments on adjoining land to defray a portion of the costs of such roads may be justified.

10.—Where extraordinary improvements are undertaken in the vicinity of or serving congested areas of population the increment, if any, in property valuation following the improvement should be drawn upon to defray an equitable portion of the cost.

11.—Irrespective of the particular form of special tax of the motor vehicle, whether registration fees or motor fuel taxes, the aggregate amount of these taxes in any one year should not be so great as to impose an undue burden on the individual motorist.

URBAN AND INTERURBAN BUSES*

BY BRITTON I. BUDD,† ESQ.

Rapid development of the bus, or as some prefer—the motor coach—in urban and interurban transportation, has given railway operators a new problem; that is, to find the best and most economical use to which the motor coach can be put, and then to fit it into its proper place in the transportation system.

The demand for motor-coach service in recent years has come directly from the traveling public. We have been living in an era of great prosperity in which the number of private automobiles averages almost one for every family in the country. The comfort and convenience of the automobile has created a demand for de luxe travel, and the motor coach has appeared to meet that demand, regardless of economic law which, under less prosperous conditions, would be a determining factor.

It is the business of transportation companies to supply the public with the character of service it demands. If the public prefers to ride on rubber tires at increased cost, the transportation company must supply that service, even if it may not be the most economical.

Co-ordination of all transportation facilities so that each may be assigned to the kind of work it is best fitted to perform is the problem which must be solved before the future of the motor coach can be forecast with any degree of accuracy. The public must look to the operators of the steam and electric railways for the solution. They are the men who by training and experience are qualified to perform this service.

That rapid strides are being made in the co-ordination of facilities is seen in the manner in which steam and electric railway companies have adopted the motor coach.

A survey made by the National Automobile Chamber of Commerce shows about 20 steam railroads operating motor coaches as a part of their regular passenger service at the end of 1925 and that 18 others are now considering similar service. The same survey shows that 51 steam railroads in the United States and Canada are using motor trucks to supplement their freight service.

The adoption of the motor coach by the electric railways of the country is much more marked. In 1920 there were only 16 such companies using motor coaches as a part of their service, whereas, at the end of 1925, 280 companies were using them.

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† Pres., Chicago Rapid Transit Co., Chicago North Shore & Milwaukee R. R., Chicago South Shore & South Bend R. R., and Chicago, Aurora & Elgin R. R.

This great increase has come in the last two years. In January, 1924, only 14% of the electric railway companies of the country were operating motor coaches. In January, 1926, the percentage had risen to 35 These companies are operating 13 000 miles of motor-coach routes.

That most of this traffic is new business is seen in the fact that electric railways in the same period carried more passengers than ever before in their history. The steam railroads carried more freight in 1925 than in any previous year and materially reduced the time lost in transit.

As the adoption of the motor coach and the motor truck by the electric and steam railroads goes on, it is probable that in the near future the existing transportation agencies will handle all this traffic as a properly co-ordinated part of their systems. The problem is one of adjustment.

Some of the electric railway companies of the country have gone into the motor-coach business on an extensive scale. The largest is the Public Service Company of New Jersey which is operating more than 800 motor coaches in connection with its rail lines. Many other companies on a less extensive scale have fleets of 50 to 100 motor coaches in service, while some operate only a few from the end of their rail lines.

This extensive use of the motor coach and the motor truck by the railway companies is only a small phase of the industry. The number of independent motor-coach operators has increased until there is a network of routes all over the United States. On January 1, 1926, 6 455 companies were operating motor coaches over routes aggregating 232 341 miles. The latest figures indicate an increase of 15% in the number of coaches and 8% in miles of route. It it probable that by the end of 1926 the miles of route will be in excess of 250 000, a mileage almost equal to that of Class I steam railroads, and the number of motor coaches will exceed 75 000.

Although the experience of the last few years has demonstrated the usefulness of the motor coach as a transportation agent, it has shown as clearly that it has certain definite limitations. The experience in some instances has been a costly one to the public and to private investors. The public suffered because of inadequate service and the independent motor-coach operator lost all or part of his investment.

In the urban field it has been shown by the experience of Akron, Ohio, and Des Moines, Iowa (to mention only two of the number of cities which tried the experiment) that the motor coach is not suited for mass transportation. It cannot substitute for the electric railway.

In the interurban field the experience of Indiana is a strong argument for co-ordination of the motor coach with the electric railway.

Comparison of the relative speeds and carrying capacity of the motor coach, the street car, and the rapid transit line (elevated or subway) under heavy traffic conditions, tends to place each agency in its proper place in urban transportation.

Under average conditions in the large city during the hours of heaviest travel, the speed of the motor coach is 8 to 9 miles per hour and the utmost

capacity of double-deck buses in a one-way movement under the most favorable conditions is from 5 000 to 6 000 passengers.

The average speed of the surface electric car is 9 to 11 miles per hour and its carrying capacity from 14 000 to 16 000 passengers.

The rapid transit lines (subway or elevated), operating trains instead of single cars, have an average speed per hour of 14 to 15 miles in local service and 18 to 25 miles in express service. The maximum capacity is from 35 000 to 50 000 passengers per hour.

In a city like Chicago, Ill., where the rapid transit elevated lines are called on to carry 80 000 to 90 000 passengers into the "Loop" each morning in the space of an hour and an almost equal number out in the same time in the evening, it will be seen at once that this service could not be rendered by gasoline-driven vehicles which occupy relatively much greater space per passenger carried.

What has been said of Chicago applies in degree to every large city in the country in the traffic peak hours. There has been considerable loose talk that the motor coach is destined to supplant the electric railway in city and interurban transportation, but there is no substantial basis on which to rest such claims. Neither are such claims made by those qualified to speak on transportation subjects.

Reference was made to the experience in Indiana. Probably no State can furnish a better example of the results of competition between motor-coach and interurban railroad routes. The fact that the motor-coach company controlling the larger number of bus routes is now in a receivership does not prove that the motor coach is not a useful medium of transportation. It proves that in Indiana it was used in the wrong way; that is, in competition with the interurban railroads for long-haul traffic. Both the rail and motor-coach lines suffered while the competitive war was on. The motor-coach lines could not compete successfully with the interurban lines on long hauls, and the electric lines could not compete with the motor coach in local traffic.

In a recent public statement Fred I. Jones, Receiver for the motor-coach lines, was quoted as follows:

"It is becoming apparent, it seems to me, that the operation of motor buses be co-ordinated with existent steam and electric forms of transportation. This is particularly true in Indiana, which is covered by such a network of steam and interurban railways serving the population fairly satisfactorily. The buses find it impossible to stand alone, but are logically indispensable in rounding out a complete and unified transportation system".

Certain definite conclusions may be drawn from the Indiana experience. It proves that competition between two transportation systems serving the same communities, prevents either system from giving the public the best service. It proves that rates of fare that do not provide a reasonable rate of return on the capital investment lead to bankruptcy.

The Indiana experience has shown that the motor coach is neither more popular nor economical than the electric railroad for long-distance travel. It has shown that the motor coach is not a medium around which any great

system of interconnecting transportation can be built, but that there is a profitable field for its operation co-ordinate with other agencies.

Courts and regulatory commissions are now generally agreed that competition between utilities giving the same kind of service is not to the best interest of the public. In a rate-cutting "war" what the public gains temporarily in lower rates it loses in the quality of the service; in the long run it pays higher rates when the competing companies combine or one is forced out of business. The company that remains must recoup its losses sustained during the "war".

Many motor-coach operators made the mistake of establishing rates of fare on a competitive basis with electric railways. Motor-coach service costs more to produce than electric railway service and it probably always will; but the public demands this special service which more nearly approaches the comfort and convenience of the private automobile. That the public has shown its willingness to pay a higher rate of fare, is seen in the patronage given the motor coach in New York, N. Y., Chicago, St. Louis, Mo., and other large cities, where higher fares are charged than on the electric railways.

Although no general rule can be laid down for every situation, either with respect to motor-coach operation or the rate of fare to be charged, experience indicates that in city service the rate necessary to maintain motor-coach service should be approximately twice that paid on electric railways. In interurban service, the rate should be from 30% to 50% more than the railroad rate. The future of motor-coach industry depends on its being made self-supporting.

California was one of the first States to adopt the motor coach on an extensive scale. At first, the industry was entirely unregulated, but afterward the motor bus was brought under the regulation of the State Railroad Commission which prescribes the system of keeping accounts. An incomplete report issued by the Commission covering motor-coach operation for the year 1924 brings up the question whether the rate structure is sound.

An analysis of the report made by the Chief Statistician of the American Electric Railway Association shows that although the California motor-bus operators carried 1.5% more passengers in 1924 than they did in 1923 and that their total revenues increased 8%, their net revenue from operations showed a decrease of 7.6 per cent. Operating expenses, exclusive of taxes, increased 10% over the previous year. Taxes were increased 165% during the year and when they are deducted the net income showed a decrease of 42.5 per cent.

The motor vehicle operators of California have had more experience than those of any other State. Nearly all of them operate independently of electric railways. If, as the report shows, they are finding it necessary to increase their operating ratio in spite of an increase in business, it indicates that the rate structure is not on a sound economic basis.

Electric railway companies which have gone into the motor-coach business have generally adapted a rate structure from their railway practice. Experience so far tends to show that railway rates applied to motor-coach transportation will not cover the cost of operation.

The largest item of expense in the upkeep of the private automobile is depreciation, an item that escapes the attention of many users. That item alone exceeds all other expenses of upkeep, including gasoline and tires, garage rent, insurance, and repairs. One large manufacturer of automobiles who has made a study of depreciation finds that the average automobile is traded in for a new model after being run 16 000 miles. The constant demand for the latest model runs depreciation charges up to a high figure.

In a measure the same applies to the motor coach. Manufacturers are constantly making improvements in motor coaches as they learn by experience. The public must have the latest or patronage will decrease. Just as the owner of the private automobile will exchange his car long before it is worn out to obtain a newer model, so must the motor-coach operator keep up with the latest styles.

With greater experience no doubt the depreciation costs on the motor coach will be materially reduced. Not enough attention has been paid to this phase of operation. Railway companies which exercise the utmost care in seeing that their railway equipment is maintained at the highest efficiency, provide few facilities for maintaining their motor coaches in the same way. They will employ only the most highly skilled workmen to inspect and repair their railway equipment, yet they will entrust their motor coaches to the care of inexperienced and unskilled men. There is no doubt that operating costs are often made higher than necessary from this cause.

If motor-coach operation is to be made successful more attention must be given to garage facilities and to proper maintenance. Careful studies must be made of tire costs and gas consumption. The motor coach must now be considered as much a part of a transportation system as the electric car and the costs of operation must be studied as closely.

City motor-coach operation and interurban or inter-city operation are separate and distinct propositions. The type of equipment that might prove satisfactory in city operation where the haul is comparatively short, would not meet the requirements in inter-city service. Motor coaches engaged in long-haul operation must be attractive in type and comfortable. Where they are in constant service on regular routes, it may not be necessary that they should be quite as luxurious in furnishings as the types used for special tours, but they must be greatly superior to the ordinary type used in city service.

In interurban and inter-city service motor-coach schedules must be arranged with great care. They must provide for convenient rest stops and public comfort facilities. These conveniences must be counted in the capital investment and in the cost of operation and maintenance, items which many independent operators do not consider.

One phase of motor-coach operation that is proving attractive to the traveling public and profitable to the operators is the chartered coach business. To a large extent this is pleasure riding. A small organization plans an outing. It may be for a single day, for a week, or a month. The motor coach is the convenient agent for such travel because of its flexibility. The time spent on the road is not a factor as in the ordinary business trip. More oppor-

tunities are afforded the traveler to see the country through which he passes. The rate which such parties are willing to pay makes the operation profitable for the carrier as the travelers are more interested in getting the kind of service they wish than in what it costs them. If the parties are of sufficient numbers to equal the number of seats, or nearly so, the rate charge can be made quite reasonable and still give the carrier a fair profit. The writer knows of instances where such chartered motor-coach operation has proved sufficiently profitable to offset the losses sustained in operating non-paying regular routes, so that the business of a company as a whole showed a margin of profit.

In summing up the motor coach situation in the urban and interurban service, some fairly definite conclusions may be drawn:

1.—In city service the motor coach has its greatest economic value when operated in conjunction with electric railways. It is a most convenient agent to give transportation service along boulevards and parks and to serve as a feeder to electric lines in territory not otherwise served.

2.—In the suburban and interurban fields the motor coach is most useful for comparatively short hauls of 20 miles or less. In long-haul traffic it is not as useful nor as economical as the high-speed electric railroad and should not be operated in territory served by rail. As an auxiliary to the railroad the motor coach has its greatest usefulness in the interurban service, as it can be used to serve territory contiguous to the railroad for a distance of 25 miles or more.

3.—The cost of operating motor-coach service is greater than that of rail service and is always likely to be so.

4.—Railway operators with their special training and experience are best qualified to operate motor coaches and co-ordinate them with the railways.

5.—In the near future, under a properly co-ordinated transportation system, the motor-coach business will be placed on a sound economic basis. Where motor coaches can be operated economically they will be run, and where it is found that the public will be better and more economically served by rail lines, the latter will carry the traffic.

THE LOGGING AND LUMBERING INTERESTS OF THE PACIFIC NORTHWEST

A SYMPOSIUM*

* Presented at the Summer Meeting, Seattle, Wash., July 14, 1926.

THE ENGINEER IN THE LUMBER INDUSTRY

By J. J. Donovan,* M. Am. Soc. C. E.

Sawing boards seems a simple task. It was in the old days of the single circular saw which replaced the primitive whip-saw and when the market was usually within the radius of one day's journey by horse from the mill. Those days are gone. Now turbines generate electricity which drives the great band-saws and the many varieties of special machines with a speed and accuracy unknown a few years ago. With the old machines went the long hours, and the mills and camps of the Northwest have had the eight-hour day for more than eight years.

Problems of Production

Lumbering is the great industry of the Pacific Northwest. The forests of Washington will last for fifty years and those of Oregon for one hundred years at the present rate of cutting, allowing nothing for new growth. The industry is here to stay—if the common sense of the engineer is applied to the problems before it is too late. What are the problems?

First.—Cutting the trees, hauling them to the mills, and sawing them into lumber without unnecessary expense or waste.

Second.—Explaining to distant markets the special value and practical use of different species and grades of lumber.

Third.—Shipping and delivering lumber from the mill to the ultimate distributor with the utmost economy.

Fourth.—Protecting the young forests in order that future generations may have fuel, implements, shelter, and clothing, such as have served man from the earliest ages.

There are many subdivisions of these four major theses, but only a few will be discussed in this paper.

Modern lumbering requires the trained mind of the civil, mechanical, electrical, and steam engineer, with skilled mechanics of many kinds, with salesmanship that grasps the need of both buyer and seller, with knowledge of the strength and life of materials. Finally, future foresters must have knowledge of silviculture and some ability to deal with the ever-present political factor.

Methods—Old and New

Until 1890 the logging on Puget Sound and along the Columbia River was almost entirely over skid-roads made by embedding maple, alder, or hemlock logs, 8 ft. long, spaced 8 ft. or less from the shore to the forest. The logs to be hauled were barked to give a smooth riding surface; the "greaser" swabbed his skids; and six to ten pair of oxen with much straining and a wealth of picturesque language on the driver's part moved the log to the water.

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Two miles was the ordinary limit for such roads. Horses occasionally replaced the patient ox (Fig. 1), but were less reliable in mud and swamp.

Following the use of horses in logging came the small single-drum donkey engines with a "line horse" struggling through the brush from the skid-road to the log. The "line horse" hauled the line from the engine drum through an almost impassable tangle to the log that was to be hauled in. The maximum range was 300 ft. The adoption of double drums and the increased use of railways (Fig. 2) came thirty years ago; but it is only ten years since a revolutionary change occurred in the handling of logs. This was the use of trees as masts (Fig. 3) with great blocks, some weighing 1 200 lb. and hung 180 ft. from the ground. The engine is run up against the tree, the main line runs through the roof to the block, and thence off to the logs. The tree or mast itself is guyed with ten or more heavy steel cables and the whole trimmed, rigged, and made ready for work with phenomenal speed by a small crew of highly skilled men.

The most picturesque figure in the woods is the "high rigger" who, with life-belt, spurs, and axe, climbs a tree to a point nearly 200 ft. from the ground, trims it, cuts off the top, hauls up blocks and lines, and ultimately rigs the tree as shown in Fig. 2. This method, used with a radius of 600 ft., immediately brought a lifting strain on the end of the log, and it came in at twice the speed of the old ground-yarding method.

Steam Is Main Power

The modern unit consists of an oil-burning boiler mounted on a heavy steel railroad car supplying steam for a duplex loading engine and a yarding engine with one main and three auxiliary drums. Each engine is arranged to run independently, yarding and loading going on simultaneously.

A good logging outfit thirty years ago cost \$25 000; to-day, it costs \$500 000. Most camps use steam generated by oil as their main power. A few use electricity. Diesel engines are in the experimental stage. Many auxiliaries are gasoline or gasoline and electric.

The latest development in logging is the heavy skidder (Fig. 4) with an articulated steel mast 100 ft. high, which, with yarding and loading engines, blocks, and lines, complete on three cars, represents an investment of \$75 000.

Such a unit can be installed with remarkable speed, the steel mast replacing the tree of the high-lead system. A tail tree, carrying the outer end of the trolley line at a distance of 1 500 to 2 000 ft., marks the radius of action. In rough ground a unit of this kind will effect a saving of about \$1 per 1 000 board-ft. over the high-lead system. This saving is due to a reduction of about 50% in railroad branches and the ability to reach into and across gulches at higher speed. This skidder system lifts the log by one end to a trolley line and then drags it to the railway.

Railway Development

Logging railways were scarce thirty years ago, and such as existed were poorly built, with 30 to 35-lb. rail. Locomotives weighed 30 to 40 tons.

To-day, the railway is the most necessary and usually the most expensive factor in logging operations. The weight of steel rail has more than doubled and that of equipment, tripled. The modern logging railway has 60 to 70-lb. steel rails, heavy geared and compound Mallet locomotives, and high-class special cars with all modern safety appliances. Special types of locomotives and cars have been developed to carry the increasing loads over the longer distances with economy and safety. On no trans-continental railway is more care used on grades and curves than on the logging railways. Fig. 5 shows a Baldwin Mallet locomotive for main-line work, which handles 24 loaded cars up a 1.5% grade. Its weight on the drivers is 90 tons.

Another type is the Climax geared locomotive (see Fig. 2) which works successfully on grades up to 8 per cent. Both engines have superheaters, which reduce the fuel consumption one-third. A typical development of a mountain side is a main line using a 6% sustained grade from the valley, a distance of more than 6 miles. Branch lines are extended from the main line on approximately parallel lines 1 200 ft. apart horizontally and from 200 to 300 ft. vertically. Thirty-six spar-trees were used on this development of 1 200 acres. Two high-lead machines and one skidder worked on this location with an average monthly output of 7 000 000 board-ft. and a cost exclusive of stumpage, taxes, overhead, and depreciation of $8 per 1 000 ft. of logs.

Primitive Bridges

To illustrate the work of the engineer in logging operations a view of the Cavanaugh Creek Bridge is submitted (Fig. 6). This structure is notable for three things:

(a) It is built almost entirely of timber cut on the site.

(b) Piling is all single length, and in some of the falsework it is 140 ft. long.

(c) The Howe truss is made of four single fir trees, 108 ft. long, braced properly, and was swung into place with an ordinary pile-driver. The height from rail to the water is 130 ft. The span was constructed to insure against slides up the canyon and sudden rushes of water, such as wrecked a locomotive and cost four lives only 10 miles away. The length is 1 000 ft., and the cost was $30 000.

The civil engineer has made his mark in the woods and his mechanical brother has kept pace at the mills and docks. Labor-saving devices of many kinds—package handling, conveyors, steam and electric cranes—all do their share to give the men of the Northwest the best pay and the shortest hours of any lumbermen in the world.

The necessity for application of engineering principles to the proper utilization of wood was never greater. Obsolete and expensive specifications are still used on many contracts. Much progress has been made in the standardization of certain sizes, varieties, and grades under the lead of the U. S. Department of Commerce.

With it all is the effort to eliminate waste and to use the right size and the right grade of lumber in the proper place. The testing laboratories at

FIG. 1.—HORSE LOGGING.

FIG. 2.—LOGGING RAILWAY.

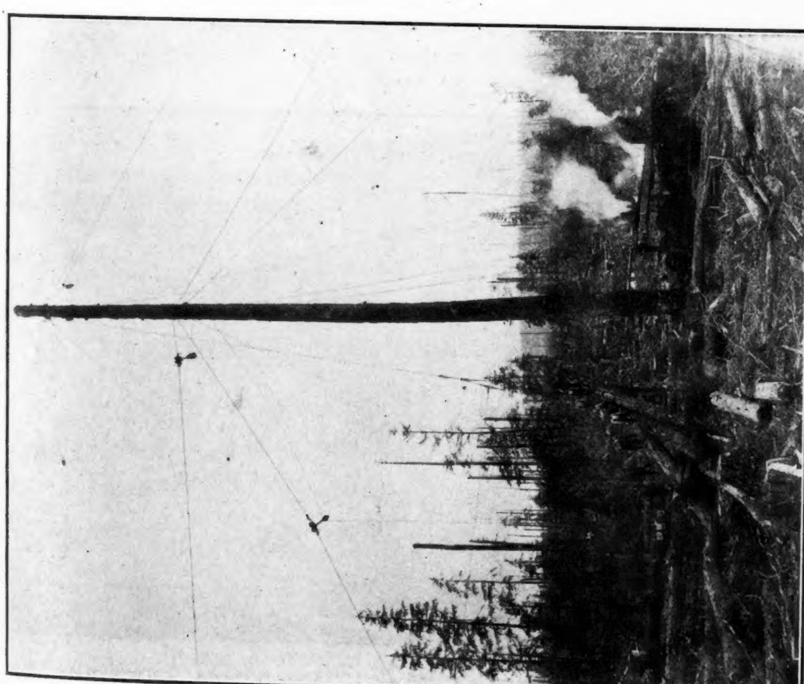


FIG. 3.—TYPICAL HIGH LEAD.

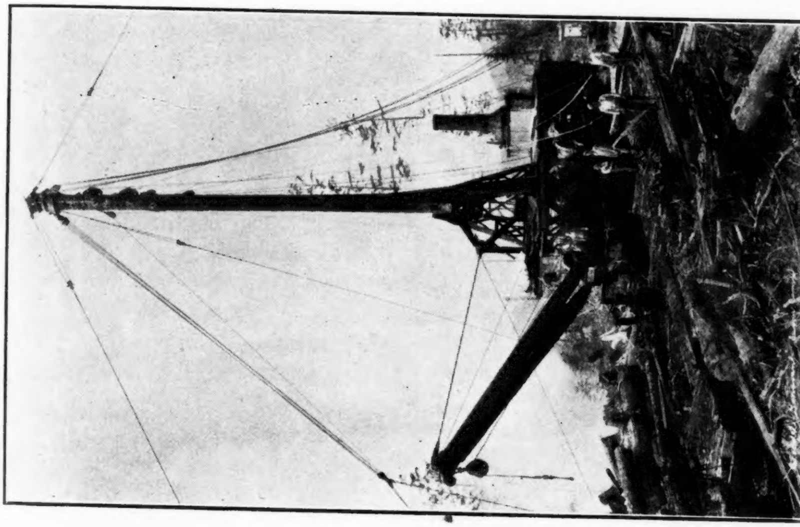


FIG. 4.—VIEW OF MODERN SKIDDER.

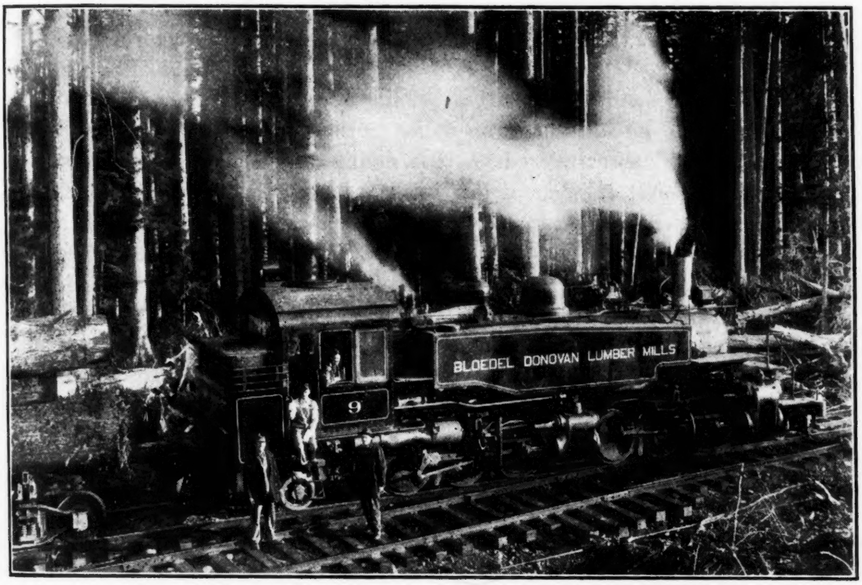
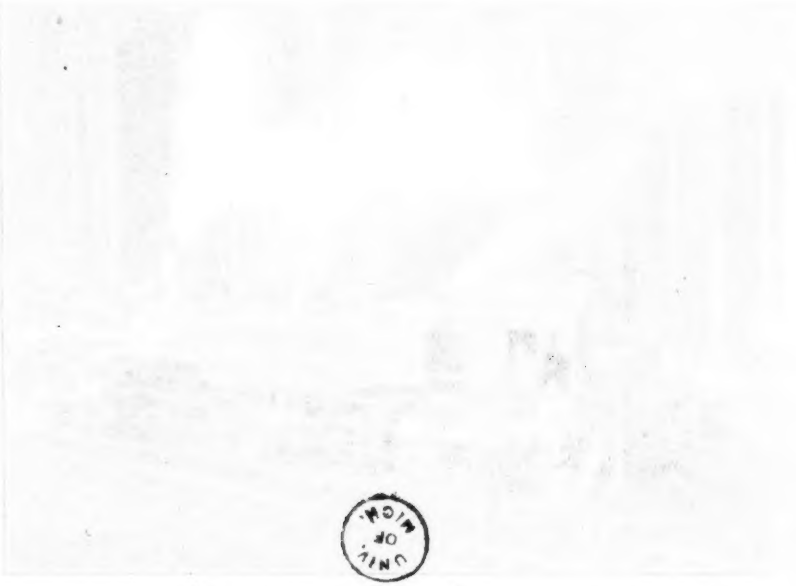


FIG. 5.—MALLET MAIN LINE LOCOMOTIVE FOR LOGGING RAILWAY.

FIG. 6.—VIEW OF CAVANAUGH CREEK BRIDGE.



Madison, Wis., under Government engineers have rendered service of great value in proving strength, fire-resisting power, and other qualities under varying conditions. Likewise, the National Committee on Wood Utilization has done notable work in bringing together the various interests and agencies dealing with wood.

There is much ill-informed talk of a timber famine. There is more of fire dangers in wood construction which do not exist and a determined, well financed, and specious propaganda for wood substitutes. The fact remains that there is plenty of wood at prices which make it the building material for nine-tenths of the homes, and for most of the granaries, warehouses, and miscellaneous buildings of the country.

All the railway ties, a considerable amount of car, bridge, and building material of the railways, is wood and will continue to be, because it gives the best service for the money. There is no substitute for wooden ties. Close and intelligent studies of methods of logging, milling, selling, shipping, and using are being made.

Reforestation

Reforestation is largely a matter of protecting Nature in its work. The National Government set aside great areas thirty years ago. It owns more forest area in Washington than is in private hands, and nearly as much stumpage. The State also has great forest areas in trust for the schools and about \$20 000 000 has been realized from sales that have been made. Washington is slowly developing a forest policy, but it still has far to go. It has more logged-off lands than any other owner. It cannot sell them because of the minimum price of \$10 per acre fixed by law. These lands are usually Sections 16 and 36 in each township. They are often surrounded by private logged-off lands, some of which are reverting to the State for taxes.

Private reforestation is impossible economically under the existing law. Even with better laws the inducement is small. The State should survey and grade its logged-off lands, sell all that will bring the minimum price of \$10 per acre for agriculture and grazing and buy adjacent lands more suitable for forests for a maximum price of, say \$5 per acre, and develop and protect the young forests which fifty years hence will have value. Some exchanges with private owners may be possible as has already been effected by the State with the National Government.

Co-Operate for Forest Protection

There is need of close co-operation between National, State, and private owners of timber for fire protection and reforestation. There is better understanding to-day than ever before and fire protection is especially good. The proper building up of State forests has only begun, not only in Washington, but in many other States. Private forests cannot survive in many States under the existing laws and are capable of giving financial return at present only as pulp producers in conjunction with paper mills.

It is well to remember that in 1907 the country produced and used 45 000 000 000 board-ft. of lumber; that in 1925 this fell to 35 000 000 000, and that per capita consumption fell from more than 500 to 300 board-ft. Further, it is significant that all the soft woods exist in great abundance and at low prices, and that engineers who produce and sell these woods ask their brethren in the Engineering Profession to remember that wood is conomically available for many classes of construction and often has special advantages worthy of careful consideration.

LOGGING RAILROADS

BY WALTER J. RYAN,* M. AM. SOC. C. E.

This paper gives a brief description of the location, construction, and operation of logging railroads, in comparison with standard railway practice, and shows the relationship between the railroad and the logging operation that it serves. It tells of the work of the engineer in connection with logging railroads and points out a field for further usefulness.

The primary function of a logging railroad is the transportation of logs from woods to saw-mill. This service was originally performed by teams of oxen that dragged the logs over poles laid across the roads as shown in Fig. 7. The modern logging railroad has progressed a long way toward standard railway practice, but it still bears some of the characteristics of this ancestor, the "skid-road".

LOGGING ENGINEERING

The first logging railroads were built without any technical engineering. This is well illustrated by Fig. 8, which shows an early type of crib bridge. It was part of the duty of the logging superintendent to locate his railroads, and he had little time or opportunity to plan further ahead than the section in which he was working. These roads were extended from year to year, until the easy country was logged off or some serious obstacle to further progress was encountered. About this time, locating engineers were called in to solve the difficulties, and the original lines had to be abandoned and new lines built to reach the tracts of timber farther back. These engineers were given no opportunity to learn the requirements of the logger and many times the results were not satisfactory. There finally came a realization of the fact that an engineer was required who understood logging. The Pacific Logging Congress, in 1912, began an active campaign for "the creation of the profession of Logging Engineering as a distinct branch of mechanical science". As a result of this effort, the Western universities have established courses and are graduating classes each year with a degree in Logging Engineering.

The larger companies now maintain Engineering Departments, and there are a number of engineering firms that do all such work for their clients, usually the smaller operators who do not feel justified in maintaining an engineer in continuous employment. There are many parts of the logging operation in which an engineer is called upon to assist, but the principal part of his duty consists in the location of railroads.

RELATIONSHIP OF RAILROADS TO LOGGING

The object of logging railroad location is not to provide a route through a territory or to connect two definite points, but to serve a given area with a

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network of rails. Any part of a tract of timber that is not within "yarding distance" of a railroad is in the same position as a piece of land above the ditch on an irrigation project. This "yarding distance" is the distance that it is economical to bring logs from the stump to the "landing" where they are loaded on cars. It varies from 600 to 3 000 ft. with different types of logging equipment in use. Thus, the spacing of the railroads will vary from ¼ mile to 1 mile.

It is the relationship between the cost of logging by different methods and the cost of the railroad, that determines the type of logging equipment best suited to each situation. As railroad construction becomes more difficult, more expense is justified for logging equipment and labor to give longer yarding.

Every phase of railroad construction and operation is intimately connected with the logging which it is to serve, and a reconnaissance for a railroad is incomplete unless it covers a study of the area from the point of view of logging as well as of railroading. The modern timber cruiser, after examining a tract of timber land, makes a detailed report of the timber that he finds, by species and grade, and prepares a contour map of the area. These cruisers' maps are made by pacing distances and by measuring elevations with an aneroid barometer. The maps are usually on a scale of about 6 in. to the mile, with contour intervals of from 25 to 100 ft. The U. S. Geological Survey has mapped part of the timbered area of the Western States, but there are some districts of which there are no maps worthy of the name.

The maps available are made the basis of preliminary study, but most operators recognize the value of an accurate contour map so that the reconnaissance survey usually takes the form of a mapping party. Well made maps, with accurate cruises of the timber, allow the operation to be planned so that the railroads can be built in the proper location for economical logging, with the minimum of cost.

Logging Railroad Location

When the logging railroad leaves the timber and is built for some distance to a log dump at the mill or rafting ground, the selection of a route does not differ from that for standard railway or highway practice, except that controlling rates of grade and curvature may have been determined by conditions in the timber (Fig. 9). This route to the timber and one or more lines through the tract make up the main line over which full trains are handled. Grades as high as 4% and curves as sharp as 24° are used on these main lines, and the construction cost runs as high as $50 000 per mile.

Branches and spurs serve the logging, and from these the loaded cars are switched by lighter locomotives. Geared locomotives permit the use of curvature as sharp as 40° and grades of 6% on these spurs. Where timber is still out of reach, inclines are used.

The field work of location is usually done with small parties of from three to six men. A transit or compass preliminary line is run, often with an axeman, a chainman, and an instrumentman, using a 500-ft. tape and setting

FIG. 7.—OXEN DRAGGING FIR LOG, 14 FEET IN DIAMETER, ON A SKID ROAD.

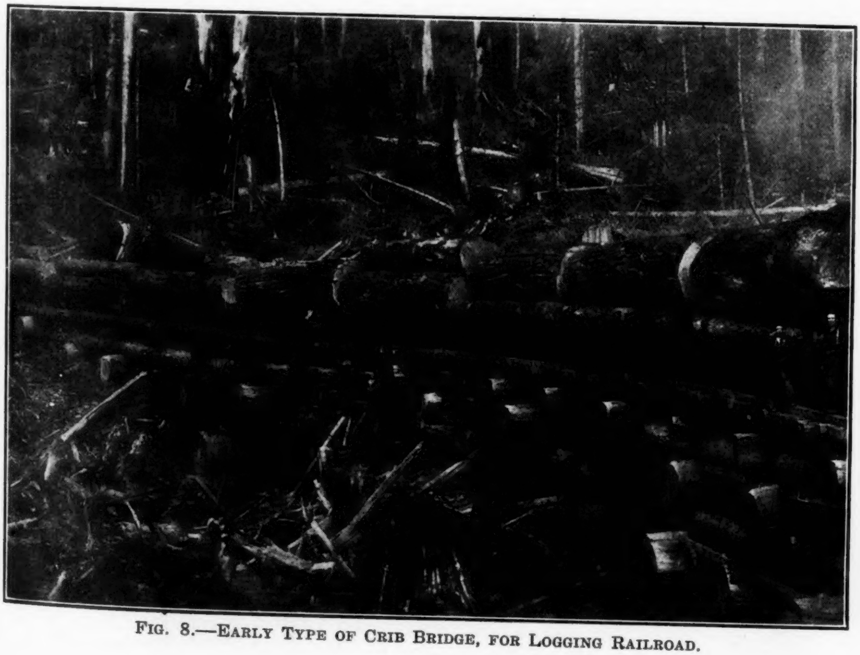

FIG. 8.—EARLY TYPE OF CRIB BRIDGE, FOR LOGGING RAILROAD.

FIG. 9.—VIEW SHOWING LOGGING RAILROAD NEAR FOOT OF MT. BAKER, WASHINGTON.

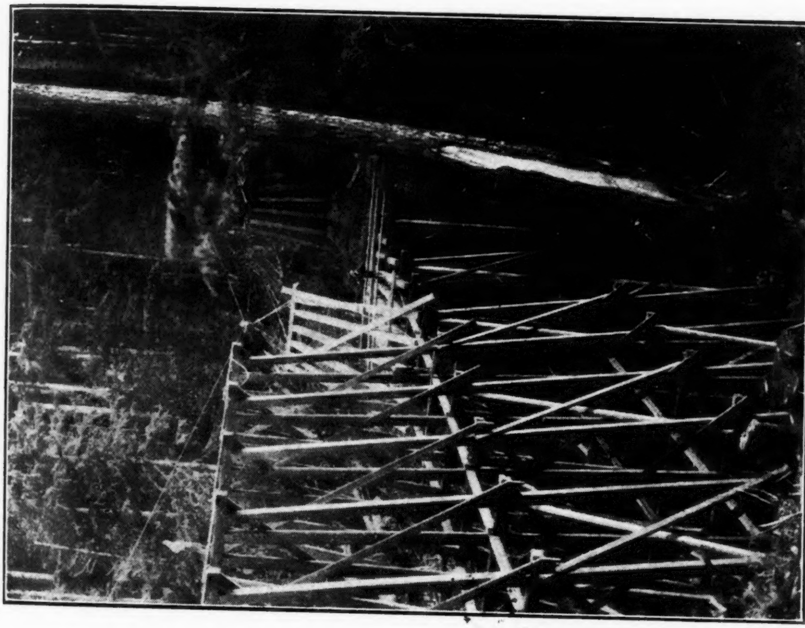

FIG. 10.—FRAME TRESTLE OF LOGGING RAILROAD ERECTED BY "SKY LINE."

stakes at 100-ft. stations. The same party runs levels and takes topography. A center line projected from this preliminary line may be run in the field or simply staked by offsets. A small party can do the locating and look after the construction of 10 or 15 miles of road per year. This proves more satisfactory than employing a larger party on intermittent work.

Logging Railroad Construction

Clearing the right of way is often the most expensive part of the railroad construction. Much road has been built with no other power than dynamite, where stumps and logs up to 10 ft. in diameter were to be removed. Horses and steam-logging "donkey" engines have been used for the clearing. Recently, gasoline "donkeys" and tractors have come into use. If power shovels or drag-lines are used for the grading, the same machine often does the "chunking out".

Culverts are usually built of logs. Where long life is required, Western red cedar or redwood is used, as these woods resist decay almost completely under the conditions found in most culverts.

Until recently, the grading was done largely by hand. The only power commonly used was a drag scraper pulled by a logging engine. A limited use is made of large steam shovels. Nowadays the small revolving power shovel or dragline, on crawler treads, is doing a large portion of the work. If the range of the machine is sufficient it is common practice to waste the material from the cuts and to borrow for the fills. The shovel runner and his helper may constitute the entire grading crew and the finishing may be left for the crew that lays the track. Where it is necessary to haul the excavated material, it is customary to use car and track, although auto-trucks are used to some extent.

The construction of bridges has shown more variation from standard railway practice than any other feature of logging railroads. The bridges that were built on the skid-roads of "bull-team" days were cribs of logs with a solid deck. The first railroads were laid over this same type of bridge (Fig. 8) with ballast under the ties. The logs in these bridges were seldom salvaged; this fact made the cost excessive when timber became of greater value. Framed trestles (Fig. 10) and pile trestles, as now used, show wide variations in design.

Pile trestles 100 ft. or more in height are frequently built. Where the ground allows little or no penetration for piling, the sticks are set, butt down, on the rock or hardpan and held in place by the driver until capped and braced. Piling as long as 100 ft., with a top diameter of 14 in., has been used in this manner.

The trestle over Cedar River Canyon (Fig. 11) is a good example of a high pile structure. This was built in two decks with Douglas fir timbers of 120 ft. and 80 ft. in length. In all, more than 500 pieces of piling and 400 000 board-ft. of sawed lumber were used, all obtained near-by. The height of 208 ft. is claimed to be the record for high timber trestles.

Pile trestles are built of material cut at the site, when it is available in suitable sizes; the caps and stringers are hewn to size and small poles are used for bracing. Fig. 12 shows such a trestle, with a log span near the center. In many stands of timber, there are no trees suitable for bridge material, and it is necessary to haul timber in by rail from the saw-mill or another part of the operation. Pile trestles are very cheaply constructed when suitable material is available on the ground. For many years, some companies built spurs entirely on trestle as they found it to cost less than grading and ballast.

Sawn timbers with depths of as much as 48 in. are available and are suitable for spans up to 40 ft. Logs are used as girders for spans as long as 50 ft., without any form of truss. Spans of 100 ft. have been built by placing a solid deck of logs on falsework and laying ties about 20 ft. in length on top of this deck. Two additional large logs are placed on top of the ties, one on each side of the bridge. The cap of the falsework is bolted through the logs above and below the ties to act as a floor-beam, and the whole structure is laced together with wire cable. On high trestles, these log spans may be used to give channel clearance and to support frame bents (Fig. 10) to carry the deck.

An overhead cableway is often used in the erection of high trestles as shown in Fig. 10. The cable is stretched above the site of the structure and on or near the center line. The cableway is used to deliver material and, in case of a framed structure, to raise the bents. Wire rope up to 2 in. in diameter is usually available around a logging operation. With the use of a logging "donkey" for power, it is possible to design such a cableway for very heavy loads. Wooden Howe trusses, steel girders, and steel trusses are in use. Their design and construction conform to standard railroad practice.

Tunnels have been built on some logging railroads and are usually timber-lined. The only variation from standard practice has been the wider section sometimes adopted to provide clearance for wide loads, due to knots or limbs on the logs. Some standard roads have refused to accept logs for haul over lines with tunnels, but there have been few accidents reported from this cause.

Track

Track laying is usually done from a push-car or from cars handled by a locomotive. A car is arranged so that the rails may be placed on the floor or bunks and the ties piled on the same car, on timbers, to hold them clear of the rail. The ties are rolled off the end of the car and the rail pulled out and placed by hand. Track-laying machines are used by some companies. These are built with an overhead trolley supported on a truss and extending about 20 ft. in front and 50 ft. behind the car on which the machine is built. The rails are piled on the floor under the machine, and two cars of ties follow behind. The ties are handled in bundles and delivered in the center of the grade ahead of the machine. When they are in place, the

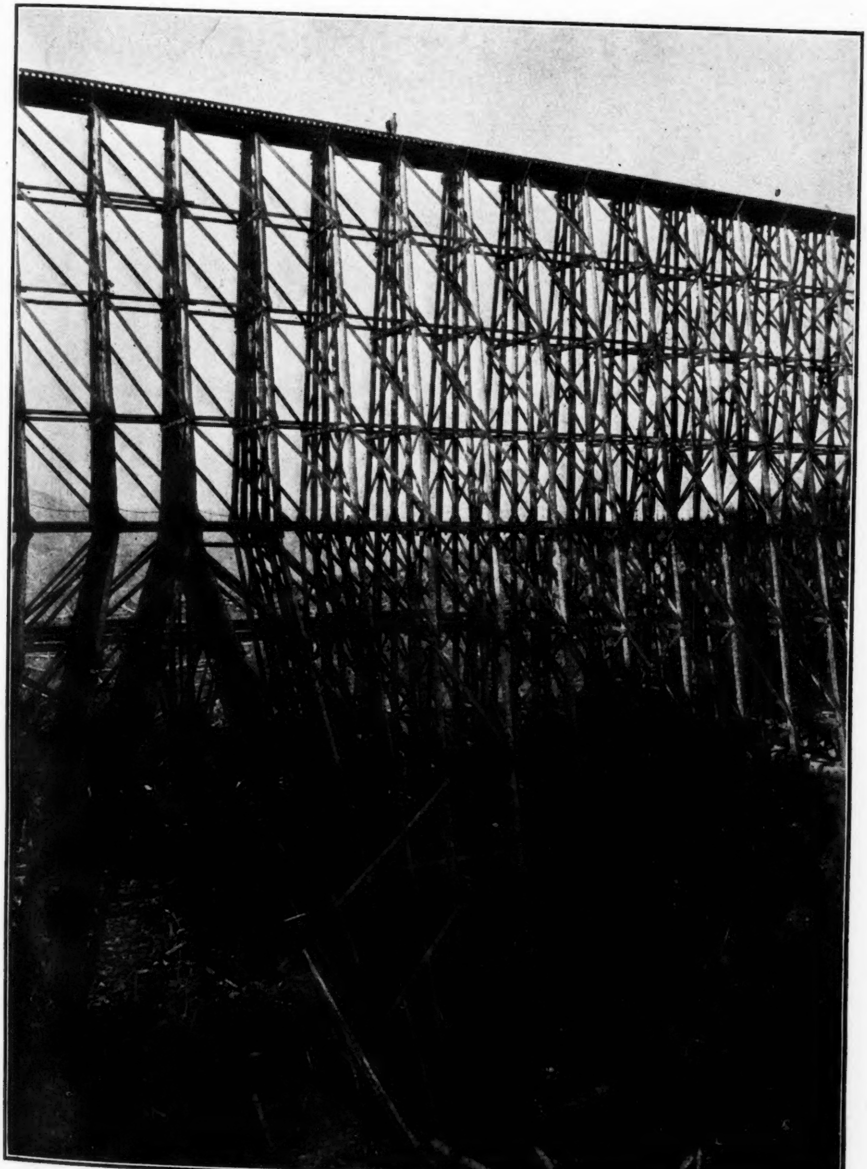

FIG. 11.—TRESTLE OVER CEDAR RIVER CANYON BUILT OF OLD GROWTH DOUGLAS FIR.

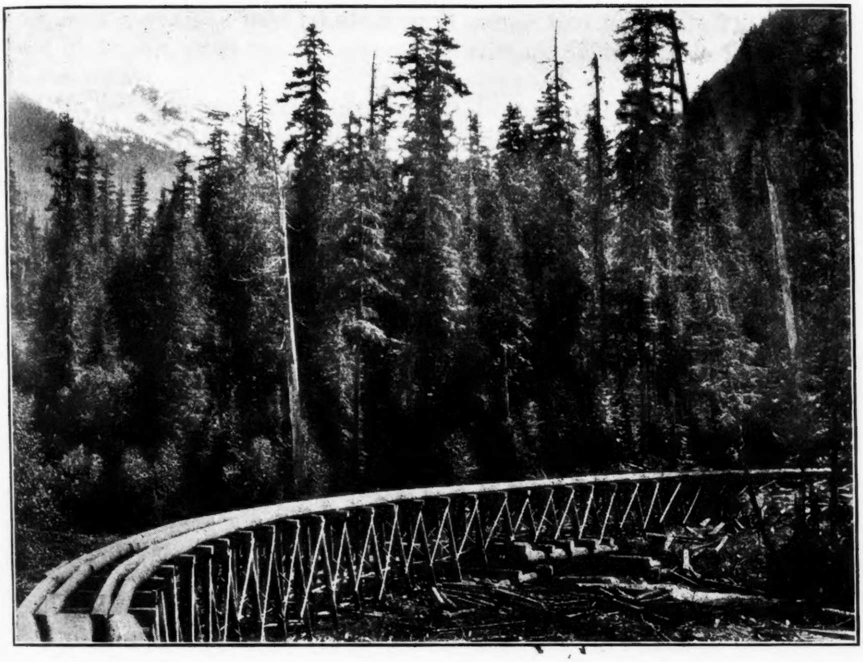

FIG. 12.—VIEW SHOWING PILE TRESTLE WITH LOG SPAN, FOR LOGGING RAILROAD.

FIG. 13.—LOGGING RAILROAD: VIEW SHOWING SKELETON LOG CARS LOADED WITH FIR TIMBER.

rail is delivered and held by tongs until swung into position. The rails are held by bridles until the train has gone ahead. Spiking is done by a crew in the rear.

The ties are usually from local timber, hewn in the woods to the thickness of 6 in. If the work is directly connected with a saw-mill, sawn ties are frequently used in sections from 6 by 8 in. to 7 by 9 in.

Good ballast is almost an essential along the west slope of the Cascades. Gravel is used where available. Sometimes it is purchased on cars and delivered by the standard railroads. Most companies have ballast pits at one or more places on their properties and load into center-dump cars. The cars are loaded by shovels, drag lines, scrapers, grab buckets, and other devices.

The most common weight of rail is 60 lb. per yd.; some of lighter section is in use and a few roads are adopting a heavier type. Re-lay rail, purchased from the standard railroads, has been used almost entirely, although some foreign rail and a little American-rolled new rail have been laid.

The sharp curves usually make necessary some lateral support for the track. Rail-braces are used for this purpose, and, on some roads, tie-plates alone hold the rail. When it is necessary to put locomotives with unflanged drivers around the very sharp curves, five rails are used, two inner guard-rails and a rail outside the track rail on the inside of the curve.

Creeping rail presents a serious problem to logging railroads. The loaded traffic is all in one direction, usually down grade. The common practice has been to allow the rail to creep and to place "points" in the track to prevent buckling. The gaps that occur are filled with "Dutchmen", and the broken angle-bars replaced. Some effort has been made to hold the track by driving piling in front of long ties and by tying the track to stumps with pieces of cable. Rail anchors have proved successful in some instances and are coming into more general use.

Train Equipment

The cars originally used for hauling logs were separate trucks equipped with log bunks. The ends of the log are placed on a pair of these disconnected trucks and these loads connected into trains. With light logs, the load is sometimes fastened to the bunk with chains, but in the fir district there is seldom anything to hold the trucks together except the weight of the logs on the bunk. Hand brakes are provided on each truck.

In order to allow the use of the air brakes, these trucks are connected into skeleton cars (Fig. 13) by means of a center sill that carries the air cylinder and brake equipment. These skeleton cars are built in lengths up to 70 ft. The standard railroads are building some skeleton cars, although most of the logs that they haul are loaded on standard "flats", provided with log bunks on the deck.

All the Western States have safety requirements governing the construction and operation of logging railroads, but these requirements are not as stringent as those for common carrier roads and allow the operation of trains

without air on the cars. A skeleton car for common carrier roads is built with platforms on the ends and a wider walkway from end to end.

A geared locomotive is commonly used for short hauls and switching. These machines are built in weights of 40 to 90 tons, all on the drivers. For longer haul, a direct-connected locomotive is used, the most popular type being the Mikado 2-8-2. For service on heavy grades, these locomotives are built with tanks for oil and water on the locomotive. The Mallet type is becoming popular and gives the advantage of a heavier unit with light wheel loads*. The Mallet type locomotives in use in the woods are much lighter than those built for standard service and are frequently built with saddle tanks.

Train Operation

Oil is the fuel most favored. The fire risk is much less with an oil-burning locomotive and this, with the saving in time and labor in handling, justifies its use, although the cost is higher than for coal or wood. Gasoline locomotives are built for light service, but are seldom used for hauling logs. Some logs are being handled by electric lines, and one logging railroad has adopted Diesel electric power.

The train length is usually limited to about 30 cars. The speed seldom exceeds 20 miles per hour with loads; but due to the short hauls and the facility with which cars are loaded and unloaded, it is not unusual to have a record of daily car loading in excess of the number of cars in use. The service is very severe on cars. Logs often drop from the tongs on to the cars, and rough track and frequent derailments make car repairs a large element in the cost of operation. This cost is much less with the log trucks and skeleton cars than it is with standard flat cars.

Train despatching is conducted by telephone and is usually very simple. Most companies have a man who gives all, or part of, his time to keeping the position of trains and directing their movements. Train crews are smaller than on common carrier roads and the labor cost of hauling is less.

All these factors combine to make the operation of the logging railroads cheaper than that of common carriers. The rates on common carrier roads have recently been increased in the State of Washington to a point where many of the loggers feel that they can build and operate their own roads more cheaply than they can pay freight. This is resulting in the extension of some roads to the mills or to tide-water. The logging roads are always being extended into more remote territory as the timber is cut. Of the track of a logging railroad, from 20 to 60%, is laid on temporary spurs that are in use only a few months and then removed.

The Field for Logging Engineering

In the Pacific Coast District, there are more than 6 000 miles of logging railroad in operation. The total is increasing about 5% per year. The annual replacement of spurs probably amounts to more than 25% of the total

* See Fig. 5, p. 827.

mileage. This makes the total length of railroad built annually about 2 000 miles, which is about one-half as much as the average annual construction of all the Class I railroads for additional main lines, yards, and sidings, during the past decade.

The railroad cost is often as much as 40% of the total logging cost and probably totals more than $30 000 000 per year in the Pacific Coast District, yet there is very little either in construction, operation, or maintenance that can be called standard practice.

The economics of railroad location, as applied to logging railroads, has received no study that is entitled to be called scientific. The problem is difficult, but the increasing magnitude of the industry would justify the expenditure of a large amount of effort in its solution. Accurate cost data on every part of the cost of logging, as well as of railroading, are the first requisite of such a study.

The wide variation in natural conditions and operating methods makes its difficult to formulate rules, and the rapid progress in the development of new equipment and logging methods soon renders old data obsolete. The principles that will cover all conditions must be basic and will require time studies of many features of the work.

Conclusion

The practice that now prevails is remarkably efficient. It has been the survival of the fittest and is a result of the good judgment and experience of a group of men who are typical of the best that American industry has produced. With such a background, there is every incentive for the engineer to carry on, and the next few years should see Logging Engineering entitled to a high rank in the profession.

SKYLINE METHODS USED FOR LOGGING

By K. Berger,* Esq.

The logging industry as a whole, not only employs a great number of engineers, but develops men faster along engineering lines than any other business. It does not necessarily follow that all engineers may become successful loggers or that all loggers are engineers, but it is necessary for the logger, in order to be successful, to develop quick and sound judgment along engineering lines, whether trained in the Engineering Profession or not. It is interesting to note the ingenious ways and means devised, in order to get results with the least amount of equipment which is always inaccessible and often inadequate, except wire rope, which is one of the most important requirements. The ever-present wire rope in the woods is no doubt responsible for the varied methods utilized for the transportation of logs, the most important of which are briefly explained.

North Bend System

Of the various skyline systems now used for logging purposes, the North Bend system is the best known. The chief reason for its extensive use even up to the present is its simplicity (both as regards rigging and machinery for operation,) and its usefulness for yarding, as well as for swing, that is, hauling loads of logs between two definite points.

With this system any double-drum machine may be used, as there are only two operating lines, the main or hauling line and the haulback line. A tight skyline is used, that is, a track cable is put up permanently for that particular setting. The simple carriage is equipped with double-track sheaves and a sheave in the bottom of the carriage. The hauling line is fastened and a fall-block is used to run on it. The chokers are fastened to the lower part of the carriage to this block, as is also the haulback line which runs out and through the haulback block. The haulback block is placed where the turn (the load of logs brought in) is to to be hooked on, then in and on to the haulback drum. The main or hauling line leads from the carriage through the fall-block and on to the hauling drum on the engine. The idea of the skyline is to lift the end of the turn to clear all obstructions or to carry the turn entirely suspended in the air while it is being hauled in, as well as to guide it and make inhaul easier.

The North Bend system is suitable only on even up-hill ground, as the lifting of the turn is dependent on the retarding strain on the haulback line, which, of course, must be paid out as fast as the turn is hauled in (Fig. 14). On a down-hill setting, the retarding strain on the haulback line would generate too much heat on the brake in order to lift the turn sufficiently to clear obstructions (Fig. 15). For that reason, in yarding down hill, the hauling line is fastened in the block and is led over the lower sheave in the carriage

* Chf. Engr., Washington Iron Works, Seattle, Wash.

and then through the fall-block, thus giving block purchase for lifting the load and correspondingly decreasing the strain on the haulback brake.

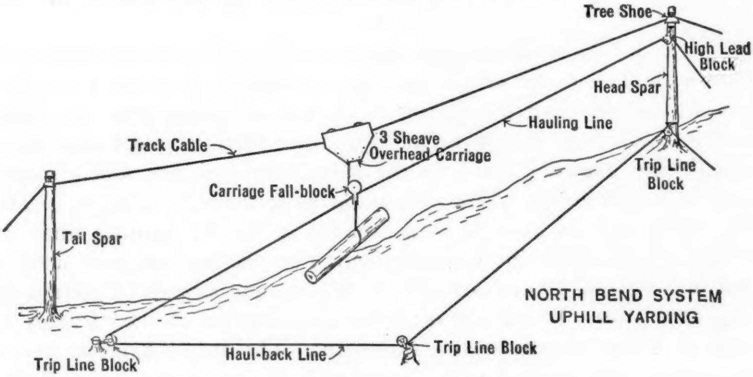

FIG. 14.

Modifications of rigging include fastening the end of the hauling line in the carriage, then leading it through the fall-block and through the lower sheave in the carriage. To keep the carriage in position while unloading, some sort of latch is utilized which serves to engage the carriage and hold it in place while slack is paid out for releasing the load.

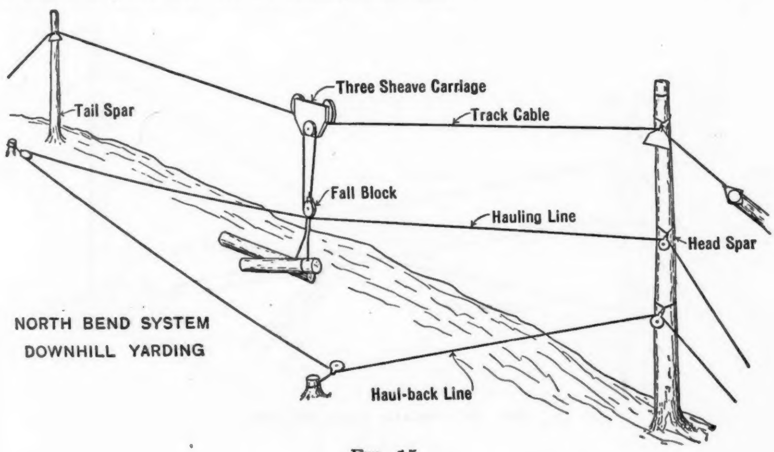

FIG. 15.

SLACK LINE SYSTEM

Until recently this system, first developed and operated in 1908, has not been used as extensively as the North Bend system. Although it is quite simple, as regards the number of lines used, it requires a somewhat larger and more complete special engine. With present-day machinery, it is one of the fastest logging systems in use, both for yarding and swinging. The skyline, instead of being permanently tightened, as in the North Bend system, is raised and lowered with each turn by a separate engine. Hauling and haul-

back lines control the travel of the carriage with its load. The chokers (pieces of wire rope with eye-splices on one end and hooks on the other end which are used for attaching the logs to the main cable) are attached to the carriage direct.

In operation, the haulback line, which is fastened to the carriage, hauls the carriage with chokers out along the raised skyline to the point where the turn is to be hooked on, then the skyline is slacked off, permitting the chokers to drop directly over the logs to be hauled, either directly under the skyline, or by placing the haulback block to the side (side-blocking) and bringing the carriage and skyline both to the side as far as practicable (Fig. 16). The skyline is raised and the turn is then brought in by the hauling line. In the early days of the slack line a simple engine or two engines were used, which did not demonstrate the advantages of this system because of certain operating characteristics. It was not until the introduction of the "Duplex Flyer" that the slack line system became successful. The duplex flyer, as now made, is the largest single-unit logging machine built, weighing, stripped of lines and water, as much as 130 000 lb. Two sets of engines take steam from a common boiler and are built on a common base. One set is used for operating the skyline, the other set for hauling, haulback, straw line, that is, a light line used for handling changes of haulback line block, and sometimes a transfer line, a heavy line for handling changes of track cable or hauling line. The engines as built are controlled entirely by air.

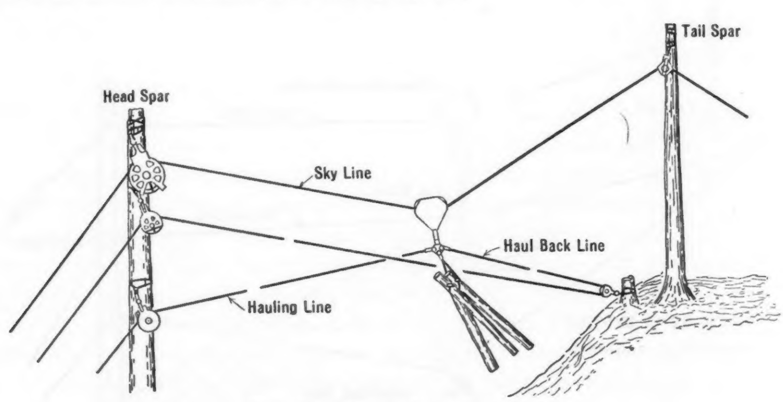

Fig. 16.—Slack Line System.

Double Slack-Line System

This method (Fig. 17), in operation and rigging, is similar to the single slack-line system, only as regards hauling and haulback lines. The skyline, instead of being fastened at the tail spar-tree, used at the outer end of the track cable, as in the single slack-line system, is led through a block at the tail spar-tree, and each end is wound on a separate drum on the engine. This permits the use of a lighter skyline and gives two speeds on the skyline inhaul. One drum may be held under brake while the other is engaged for winding in the line. This gives one speed. If both skyline drums are engaged at the

same time, the speed will be doubled and, consequently, the pull halved. This system has been used successfully for spans more than a mile in length.

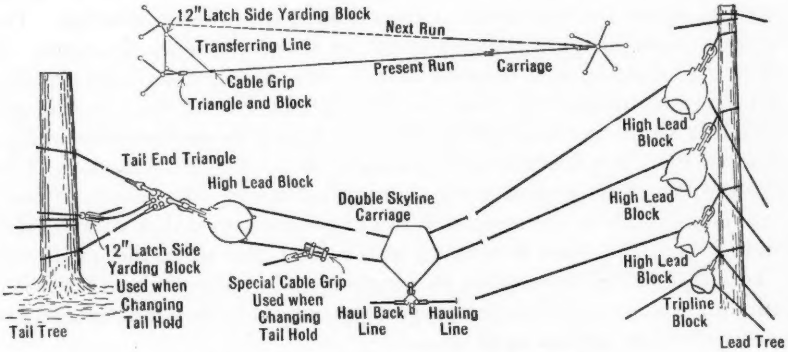

FIG. 17.—DOUBLE SLACK LINE SYSTEM.

INTERLOCKING SKIDDER SYSTEM

This is one of the oldest systems. A tight skyline and a carriage equipped with two track sheaves, one slack-puller sheave, and a block known as the depending block (fall-block), are used (Fig. 18). The skidding line leading from the skidding drum on the engine is run through a block in the spar tree and then through the depending block on the carriage. Another line, the

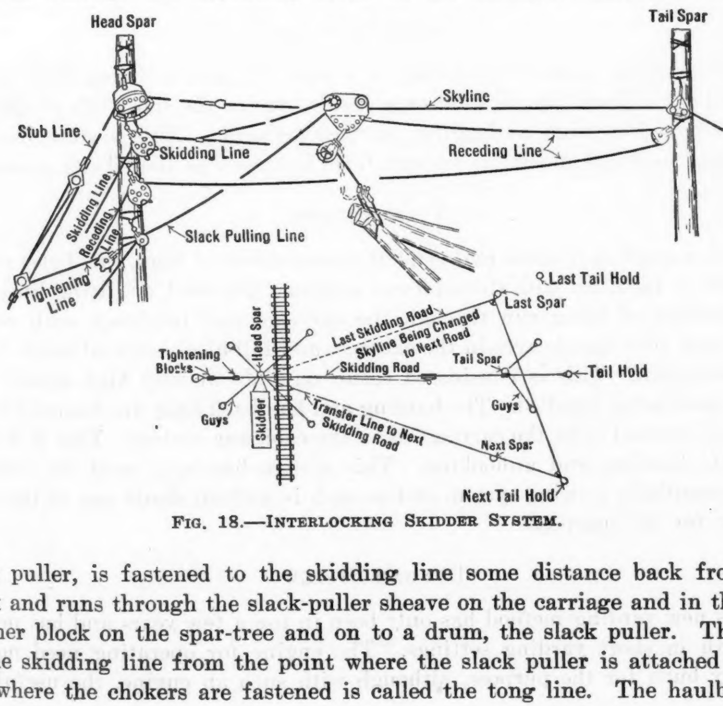

FIG. 18.—INTERLOCKING SKIDDER SYSTEM.

slack puller, is fastened to the skidding line some distance back from the block and runs through the slack-puller sheave on the carriage and in through another block on the spar-tree and on to a drum, the slack puller. The part of the skidding line from the point where the slack puller is attached to the end where the chokers are fastened is called the tong line. The haulback or

receding line, as it is called, on the skidder is fastened in the carriage and runs out to or near the tail spar-tree and returns to the drum on the engine, known as the receding drum. This drum is geared so that the line speed, when unwinding, equals the line speed of the skidding drum when winding. These two drums, when both are engaged, make an interlock, hence the name. The strain on the skidding and receding line is thus equalized on level ground as the tendency to unwind the receding line is transmitted through the gears to a winding strain on the skidding line. With both drum diameters and gear ratios alike, there is a dead strain resulting from the weight of the turn. Opening the throttle of the engine gives the required travel. These are, of course, only the main points in the system, as it will be understood that with one drum unwinding while the other is winding with a fixed gear ratio, there will eventually be a differential in speed to be equalized. To reduce this differential to a minimum is a matter of engine proportions, which follows well-defined practice as in all other skyline methods.

Another way of obtaining an equalized interlock line speed is to use two gypsy spools of the same diameter and arranged to be interlocked for handling skidding and receding lines, which are wound in opposite directions around the spools. These spools are independent of the skidding and receding drums, which are not geared for interlock. The line pull is imparted from whichever drum happens to be winding. The other drum is left to unwind with just sufficient strain on the brake to keep the line from sliding on the spool. Interlocking skidders are used both for yarding and swinging.

Cableway System

The cableway system, consisting of a tight skyline, a lifting line, a hauling and a haulback line, is used only rarely, due to the short life of the lines when stressed as high as logging practice demands. This system has been somewhat modified and in its present form is known as the "Tyler system."

Tyler System

With respect to number and general arrangement of lines, the Tyler system (Fig. 19) is identical with the cableway system. The load, or lifting line, however, instead of being run through the carriage and fall-block with reverse bends, runs over the sheaves in the carriage and fall-block with all bends in the same direction. This is considered easier on lines at very high speeds with heavy loads being handled. The hauling and haulback lines are fastened in the fall-block instead of in the carriage as in the cableway system. This is done to facilitate hooking and unhooking. This system has been used for yarding, but is essentially a swing system and as such is without doubt one of the most efficient for this purpose.

Dunham System

This new yarding method has only been in use a few years and has proved its worth in short yarding settings. The engine for operating need not be specially built for the purpose, although with such an engine, the usefulness

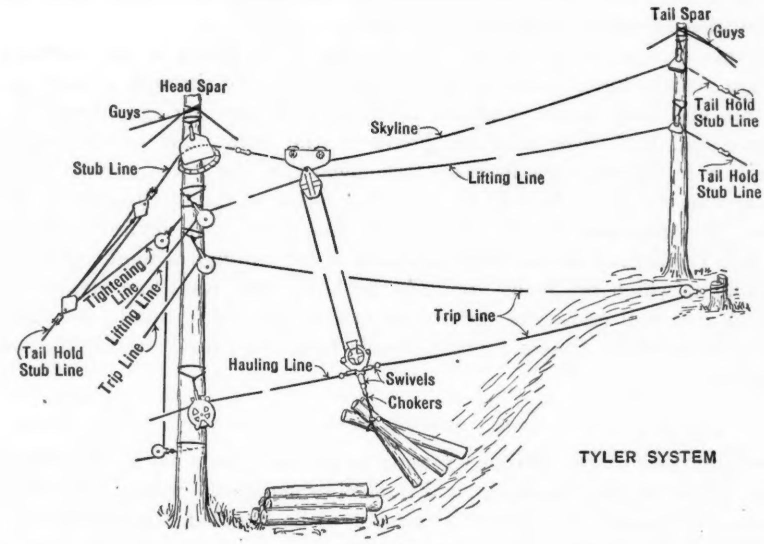

FIG. 19.

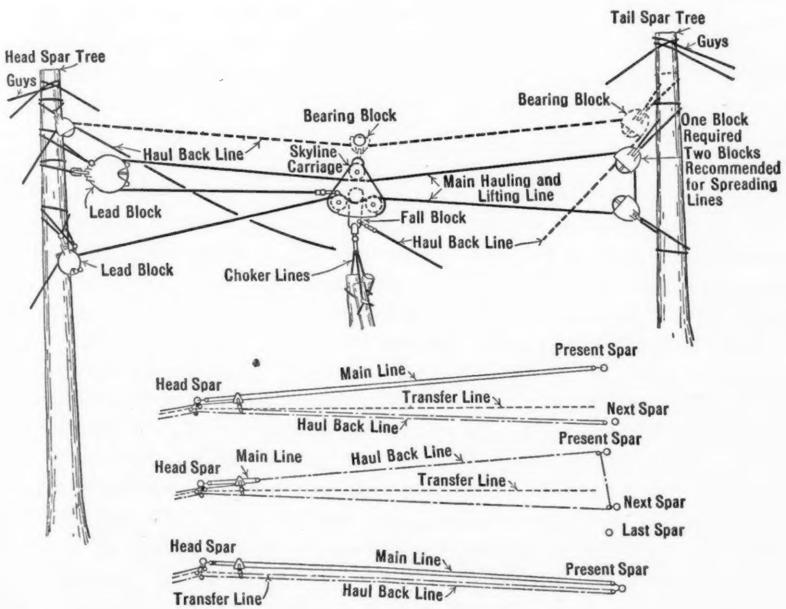

FIG. 20.—DUNHAM SYSTEM.

of the system would be extended considerably. To the present only simple three-drum machines with good brakes on two drums have been used, one of these drums having a large line capacity.

In this system (Fig. 20) the hauling line is fastened in the carriage on the edge nearest the engine. From this point it is led through a block on the spar-tree and back under a sheave in the top of the carriage, thence to the tail tree and back over another sheave in the lower part of the carriage; through a fall-block, then over another sheave in the lower part of the carriage and in through a lead-block on the spar-tree to the hauling drum on the engine. The hauling line, it will be seen, thus serves as a skyline The haulback line is fastened in the fall-block and is led through a haulback block at the point at which the loads are to be hooked. Any retarding strain on the haulback line will cause the turn to be raised from the ground. In this system, as in all other systems, modifications have been used to suit individual conditions.

Conclusions

Not any one of the skyline systems mentioned could be used exclusively because of the varying working conditions obtained in the woods. The kind of equipment at hand, the amount of timber to be handled, the contour of the ground, and the organization, all have a bearing on which system is the most economical to use. Radical improvements in the equipment for handling any one system may eliminate others, but there would still be conditions justifying the use of most systems.

LOGGING FLUMES

By Ulysses B. Hough,* Esq.

For some years the writer has been interested in the transporting of white pine logs, mining timber, and cedar poles from their place of production in Idaho, Washington, and British Columbia, to the place of manufacture and shipment to Eastern markets. Two methods have been used for transporting where such distances as 20 to 100 miles are necessary: First by the use of natural streams having sufficient water to float timber at all seasons of the year; and, second, by railroads, the former being much cheaper but not always available.

Methods of Transportation.—In using either method of transportation—by water or by rail—the products must be gathered to some central point of loading. Various means are used for this, such as hauling by sleighs in winter, the use of trucks in summer, V-shaped flumes, and sliding or trailing chutes in which two logs are placed side by side and the top hewn to form a flat V, the logs sliding by gravity on the heavy grades and being hauled over the lighter grades by horses, caterpillar tractors, or donkey engines.

Water Supply.—The writer early saw the value of the V-shaped flumes, as much of the country is rough and rolling and well supplied with small streams. Some streams afford an abundant supply of water at all seasons of the year for transporting timber, whereas others have so small a flow that impounding dams are necessary to store water for flood periods. A very insignificant stream affording only 2 to 5 sec-ft., if impounded for 2 to 12 hours, will afford an abundant supply of water to transport 100 000 to 200 000 ft. of timber many miles each day. Although the initial cost of the flumes is great, the cost of upkeep and operation is so low that it becomes a very attractive means of transporting timber.

Construction Methods.—The usual method of constructing these flumes is shown in Fig. 21. The sides are placed 45° from the vertical, forming a V, with an opening of 90 degrees. Various angles have been tried for this opening, but 90° requires the least water for floating the timber and affords a ready relief in event of jams, by increasing the quantity of water, thereby affording a greater carrying surface. A three-cornered piece, formed by splitting a 6 by 6-in. timber cornerwise, is placed at the bottom of the V to economize in the use of water and afford strengthening as well. The yokes or ribs of the flume are spaced 4 ft. on centers, and these, in turn, are supported by stringers 16 ft. long. A walk is usually provided to afford easy inspection of the inside of the flumes and access to any jam that may occur. Fig. 21 shows by circles up to 36 in., how little water is required to move the timber. This is taken advantage of in branch flumes where only 4 000 000 to 10 000 000 ft. of timber is to be removed to the main line. Although the logs may drag heavily, causing excessive wear, the flume will last through the operation which is all that is desired.

* Cons. Engr., Spokane, Wash.

Hugus Creek Flume.—To show more fully the use of these flumes and afford a basis of costs the writer will describe the flume built by him on Hugus Creek, near St. Joe, Idaho, for the Winton Lumber Company. This flume, known as the main-line flume, is 7 miles long, with three branches, 1, $1\frac{1}{2}$, and $1\frac{3}{4}$ miles long, respectively. The main Hugus Creek has a flow of only 8 sec-ft. at low stage, making impounding dams necessary. This dam is large enough to hold water for a 1-hour flood. As much as 100 000 ft. of timber is removed in 40 min. to the St. Joe River, where it floats to the mill at St. Joe, about 9 miles below.

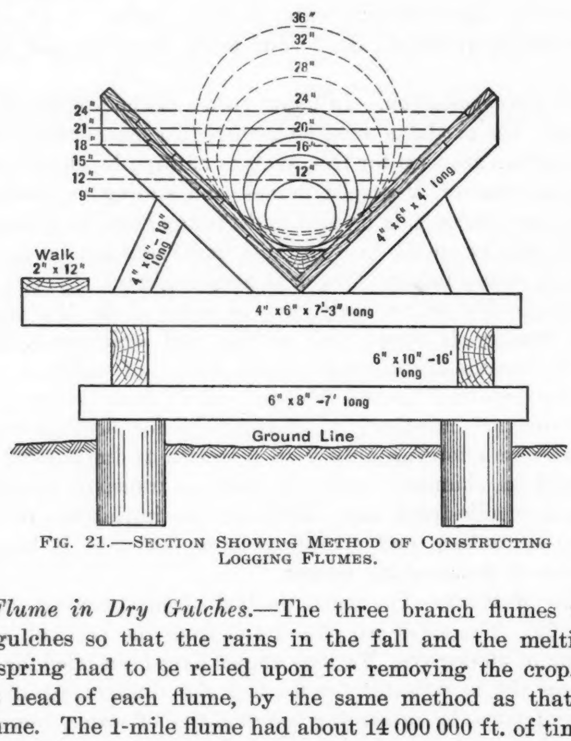

FIG. 21.—SECTION SHOWING METHOD OF CONSTRUCTING LOGGING FLUMES.

Use of Flume in Dry Gulches.—The three branch flumes were built in nearly dry gulches so that the rains in the fall and the melting snow and rain in the spring had to be relied upon for removing the crop. Dams were built at the head of each flume, by the same method as that used on the main-line flume. The 1-mile flume had about 14 000 000 ft. of timber tributary to it and 7 000 cedar poles, all of which were removed in two seasons with no trouble. The $1\frac{1}{2}$-mile flume had no water that would resemble a creek in the summer season, but 4 000 000 ft. of timber and 4 000 cedar poles were removed in one season. The logs were decked along the flume during the fall and winter and were removed the following spring in a few days. The $1\frac{3}{4}$-mile flume had 7 000 000 ft. of logs and 4 000 poles which were removed in two seasons. This gulch had a set of logging camps and the water supply was not enough to supply the camps the entire season, yet, 7 000 000 ft. of timber were removed at a cost far below any other method that could have been used.

Size of Flume.—The timber of this part of the country is generally small, 10 to 11 logs to 1 000 board-ft., although some logs in the upper part of the main line were 4 ft. in diameter. A flume with 4-ft. sides was selected for

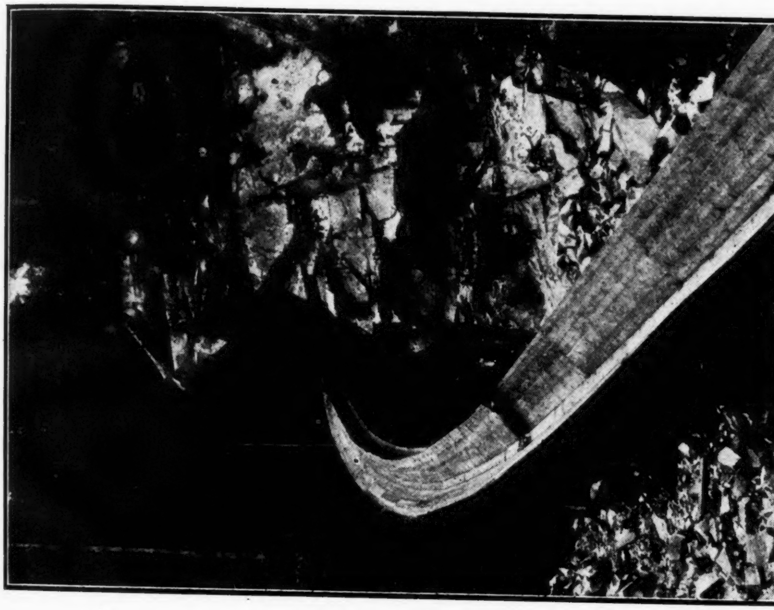

FIG. 23.—LONG RADIUS CURVE IN LOGGING FLUME.

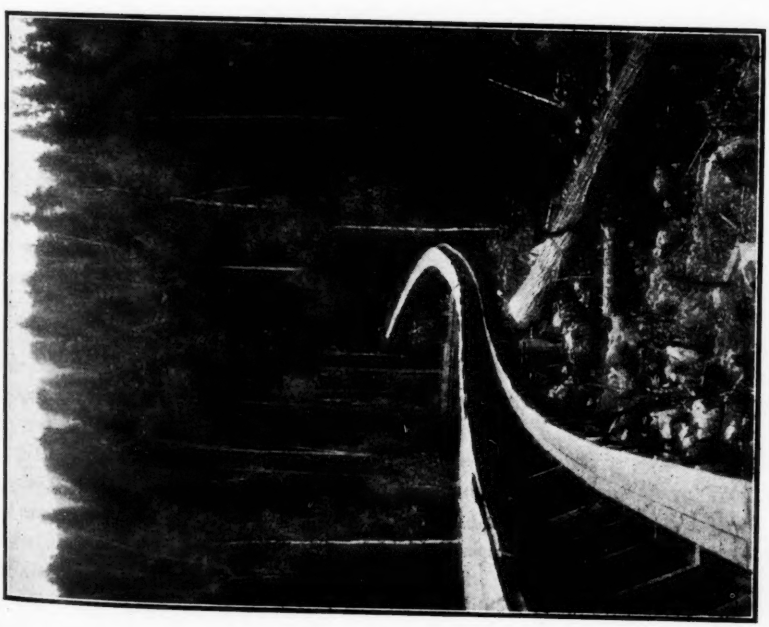

FIG. 22.—LOGGING FLUME FROM 30% TO 9% GRADE.

the main line, $3\frac{1}{2}$-ft. for the 1-mile line, and 3-ft. for the other two. All flumes are lined with 1-in., rough material doubled. Small saw-mills of capacities of 10 000 to 15 000 ft. per day were located along the main line, 2 and $2\frac{1}{2}$ miles apart, for cutting the material. The flume was started at each mill and the water of the stream diverted into it, the construction material being floated to place from day to day as the work advanced, no material however, being allowed to get on the ground after leaving the mill. Only such timber as spruce, red and white fir, and tamarack were used in the construction. The average progress per day was 320 ft., the greatest in one month being 8 000 ft. The branch flumes being smaller, greater progress was made; as much as 640 ft. was placed in 1 day and 400 ft. was an ordinary day's work.

Power.—Steam was used for power in some of the small saw-mills and in others gasoline engines were used, with the result that gas has proved the cheaper, although the gasoline was brought in on muleback or pack train and cost on the ground $0.30 per gal. Two elements entered into the use of gasoline: First, the fire risk which is almost negligible; and, second, the inaccessibility of the territory in which such flumes are used. As these flumes are often built in countries that have no wagon roads, simply pack trails, the mill and engine can be taken apart, loaded on muleback, packed to the place, and re-assembled. The writer has built one flume in which the plant has been moved 20 miles in this way. The gasoline used was $2\frac{1}{4}$ gal. per 1 000 ft. of lumber cut.

Allowable Grades.—The grades of flumes vary, being 1% at the discharge end and as high as 16% in some parts of the main line. One of the branches has a 22% grade for 800 ft. As a rule 3 to 6% grades are used. What the limiting grades of a flume should be is a question; the writer has used as high as 40% with satisfactory results. Care should be taken in the transition from one grade to the other to use long vertical curves. Although it is desirable to use as little change in grade as possible, this particular flume was intended to receive timber nearly its entire length and, therefore, had to be located on the lowest ground and near to the ground, in order that the object for which it was being built might not be defeated, namely, moving timber cheaply. Too much money should not be expended in getting timber into it.

Curvature.—Curves should be of as long radius as possible to prevent wear on the sides of the flume, 10° being the maximum used in this construction. Poles 60 to 70 ft. long pass these curves with no trouble. Fig. 22 shows a change in grade from 30% to 9% in a V-type flume and Fig. 23 illustrates the use of long radius curves in such flume.

Heavy Grades.—These flumes on heavy grades are often spoken of as "wet chutes", but the timber, traveling faster than the water by 10 to 12%, plows the water out of the flume to some extent, and, at the same time, piles it up and rides on top of it, preventing the heavy wear that occurs in wet chutes or slides.

Handling Timber to Flume.—It is often desirable to place timber in the pond formed by the impounding dam. Part of these logs can be sluiced through when the water has flowed out so that the flume will not overflow at

full-gate opening. A bear-trap gate placed in the sluiceway up stream from the sluice-gate is more convenient, and enables the sluicing of logs during the entire flood. The shorter leg of the bear-trap is placed up stream so as to control the flow of water at full head, the longer leg forming a guiding apron for the passage of logs and water to the floor of the sluice below. This trap is hinged at its upper end to the sluiceway floor and the cables attached to the apex hinge-pin pass to a winch on the top of the dam. It is better practice, however, to place the timber in the flume below the dam, and if a constant flow of water is available to dispose of it as fast as received. Otherwise, the timbers are decked alongside so that advantage may be taken of periodical floods or splashes.

Fig. 24 shows the number of logs that will pass through the flume per hour for various depths of water and grades.

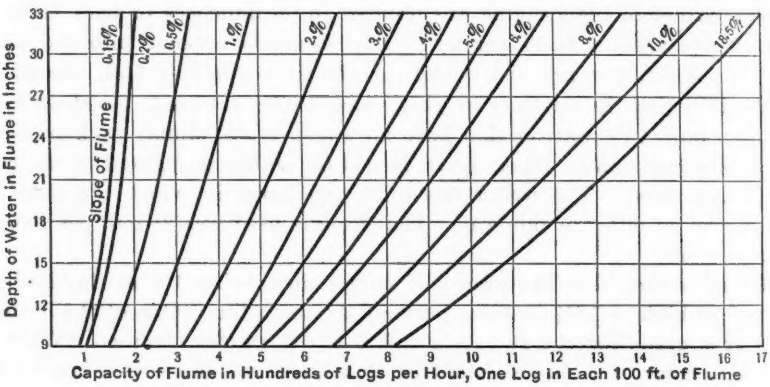

FIG. 24.

Cost of Flume.—The cost of this flume has been as follows: The 4-ft. flume mentioned cost $9 183 per mile, or $1.74 per ft. With the impounding dams, the cost was $9 733 per mile, or $1.84 per ft. of flume. The main flume has two dams, one at the head and the other 2 miles below. The wages paid were $6.00 for flume carpenters and $4.50 for common laborers for an 8-hour day.

The 1¾ miles of 3-ft. flume cost $6 466 per mile, or $1.22 per ft. of flume, the dam and flume costing $7 379 per mile, or $1.40 per ft. of flume. The wages paid were $5.00 per day for flume carpenters and $4.00 for common laborers for an 8-hour day.

The entire flume, dams and branches, cost $106 899. The timber to be removed was 60 000 000 ft. of logs, thus making a cost of $1.78 per 1 000 ft. b. m. for the flume. The cost of upkeep of the flume for 1925 was $0.03 per 1 000 ft. of logs flumed and the cost of fluming has been $0.49 per 1 000 ft. b. m. Only the flume walker was employed for the entire length of the flume, with an occasional day of extra help for repairs.

Future Possibilities.—These flumes have been used successfully for transporting cord wood, posts, poles, mining timber, lumber, and logs over long as well as short distances, and as knowledge of their reliability becomes more general, their uses will be extended.

LOGGING INCLINES

By H. G. Cowling,* Esq.

The logging incline may be defined as the connecting link between two separate and distinct units of logging.

Modern logging operations of any magnitude cover stands of timber ranging in elevation from the main watercourses to the natural timber line. This difference of elevation may be 3 000 ft., or more, depending on the topography of the country in the timber holdings.

The logging industry, like any other, has developed with the demands made upon it. The early operations were naturally along the coast line and as the timber line moved slowly away from the coast up the natural waterways to the higher levels, the method of logging necessarily changed, not only to take care of the rougher ground but to supply the increased demand. Having passed through the era of "bulls and cant-hooks", horses, road engines with horse yarding, and road engines with ground-lead yarding engines, lumbermen now use the high lead, slack skyline, and skidders. With these more modern methods of yarding and by the aid of the switchback method of railroading, the higher timber of lower unit is successfully logged.

Change of Method

As early as 1906 it became apparent that some plan had to be devised for removing timber from places that for physical or financial reasons were inaccessible to the logging railroad. Such conditions brought about the first spur-tracks too steep for geared locomotives. Thus, with an old yarding donkey or road donkey engine used as a snubber the first single-line incline was devised.

The empty car was "snubbed down", and without unhooking, was "spotted" at the landing. When loaded the car was pulled up the incline by the donkey to the top of the hill and another empty car sent down. No money was spent for additional equipment, and under the conditions at the time it probably served its purpose well in comparison with other equipment and camp output.

It is evident that such an appliance could not keep pace with the strides in logging during the past twenty years. The single line ("load down one trip and empty up the next trip") type of incline is still used in the logging of isolated settings, and in some instances this type has been refined by the use of a special snubbing engine, special line, and the necessary side and track rollers.

The first snubber of this kind was installed near Ranier, Ore., about 1906, and replaced a log chute 3 300 ft. long, with an average grade of 25 per cent. The operation of the log chute was discontinued because of the large loss in breakage. The first snubber was equipped with hydraulic cylinders to control the load in its descent but proved impractical and was soon discontinued. It was used long enough to demonstrate that the general plan of handling cars in this manner was feasible, provided the right kind of equipment was built to serve it.

* Chf. Engr., St. Paul & Tacoma Lumber Co., Tacoma, Wash.

First Snubber Built

For this purpose the Willamette Iron and Steel Works, of Portland, Ore., constructed a direct-geared engine with 16 by 20-in. cylinders and a single drum. Steam was furnished by a horizontal tubular boiler. This engine was in successful operation for many years, and probably was the first hoisting and lowering engine used in the logging industry.

The forerunner of the modern lowering engine designed particularly for the job, was the "gypsy"* type, counterbalance machine, built for the Yosemite Lumber Company, of El Portal, Calif., in which an empty car ascended as a loaded car descended. The next important counterbalance was Incline No. 1 of the Manley-Moore Lumber Company's operation at Fairfax, Wash., in 1914. Later, the writer had the opportunity of operating this incline, and studying that operation technically.

About 1913, Mr. Hugh E. Sessoms, of the Ebey Logging Company, at Arlington, Wash., conceived the idea of putting a block purchase in the line, using a pilot car carrying three 36-in. sheaves around which the lowering line passed in going from the "snubber" to the tail hold. This, with the counterbalance type of incline, will be elaborated later.

Since 1916 the development of the "snubber" or lowering engine has reached a very high order. Extensive investigations and tests have been made to determine the power consumed, the proper dissipation of the heat generated, and the economical distribution of materials. The design of the brake has been a great study as well as the character and cross-section of the brake flange. The cross-sectional design of the "gypsy" has come in for its share of study to eliminate rope wear through the side and end slipping of the rope during one operating run, and there is still room for improvement. Brakes are operated by steam, air, and hand levers, with the most recent installations using air almost exclusively.

The steam requirements of a lowering engine of the counterbalance type are usually very small, and the size of the steam plant is determined by the loads to be hoisted. In the single-line system, steam requirements are much greater. Electricity lends itself admirably to the incline problem and where available should be used. Advantage can be taken of the generating feature of the motor to assist in braking or retarding the loads in their descent.

The importance of the development of the lowering engine can be gauged from the fact that the simple early machines of twenty years ago sold for \$3 000 to \$4 000, whereas recent self-contained machines sell for as much as \$20 000 and, in one instance, an electric machine was installed at a cost of about \$60 000.

The Yosemite Incline

The first counterbalance incline of the Yosemite Lumber Company, at El Portal, measured 8 000 ft. on the slope of the track, with a difference of elevation of 3 100 ft., starting at Elevation 1 900. The foot of the incline is 54 miles up the Merced River from the mill at Merced Falls. From the top of the

* Large drum built like a spool, which carries 3½ wraps of line.

incline all timber was reached from a main line using 4% grade with a maximum curve of 40 degrees. The incline is located from the highest point of the ridge, without horizontal curves, to the center of the railroad yards at El Portal, and has gravity switching tracks at the top and bottom. The Merced River is crossed with a 76-ft. span and there are two other trestles over ravines and a bent trestle at a point where one side is over a cliff. The grades range from 10 to 78% maximum. The 10% grade is in a saddle in the ridge, and comes at the foot of 700 ft. of 78% grade. At this point on the double track, towers were erected, anchored with rock and concrete to prevent the cable from lifting 250 ft. in the air. A system of ten sheaves for each cable, set in the arc of a circle 16 ft. above the track holds the line in place and the cars pass under as they would enter the portal of a tunnel.

The flat cars used are the steel under-frame type especially designed. A bulkhead is built at one end 5 ft. above the bunk line, and riveted to the car. The cable is attached to a cast-steel draw-bar riveted to the car above the draft gear.

The first car was lowered on this incline on July 27, 1912, seventeen cars being lowered that day. The success of the operation having been proved with an initial line speed of 500 ft. per min., this was increased to 960 ft. per min. the following year, and with a few changes in vertical curves the ultimate result was the safe delivery of a car in 7 min. with 1 min. for switching and hooking on another car. This Company is now operating its second incline, 9 000 ft. long, with a maximum grade of 68%, in conjunction with a third incline about 2 miles back, 1 500 ft. long, with a 30% maximum grade.

The first incline of the counterbalance type in the State of Washington was built by the Manley-Moore Lumber Company, at Fairfax, Wash. This incline is 4 900 ft. long horizontal measurement, with a difference of elevation of 1 600 ft., beginning at Elevation 1 500. It has a maximum grade of 59 per cent.

After a careful study of this incline the writer was asked to select the proper site and build Incline No. 2 across the Carbon River from the mill of the Company. This incline is 3 323 ft. in horizontal measurement, with a difference of elevation of 815 ft. Of rock, 5 000 cu. yd. were moved, and a log bulkhead, 400 ft. long and 14 ft. high, was built near the top. The "snubber" is designed to handle 40 loads of logs per day of 8 hours. About 280 000 000 board-ft. are to be moved. There must be gravity switching tracks at the top and bottom; single track on the lower half and double track on the upper half, the cars passing in the clear above the half-way switch. A contour map was made of the strip of ground selected for the site, and a center line projected, which would place the 50-ft. contours as nearly an equal distance apart horizontally as practical, thus insuring a minimum of excavation with the most uniform grade possible.

It is the curve of the uncounterbalanced load rather than the actual ground profile that is the measure by which the operation of an incline can be judged. The "snubber" at the top of the hill absorbs the energy not otherwise consumed in friction and in bringing up the empty car. This is evident in the

form of heat at the brake drum. Points on the load curve were calculated for each 40 ft. from "hook-on" to "hook-on" and plotted as on Fig. 25, showing a profile of the incline on which has been drawn the curves for two different conditions of loading. The lower curve is for a 32-ton loaded car and a 14-ton empty, and the upper curve is for a 45-ton loaded car and a 13-ton empty.

The ascending (heavy) portions of the curve represent brake applications; the nearly horizontal portions represent a period of holding the brakes applied and the descending portions represent periods of brake releases. For example, following the descending load, note that, while the load is passing over the 200-ft. vertical curve from 11 to 45% at Station 65, the brake application increases from 3 250 to 29 800 lb., at which point there is a slight release as the load comes on the 45% grade. This release is necessary for the reason that, while the load continues on the 45% grade, the ascending empty car is changing from a 28% grade to a 34% grade on the upper end of the 160-ft. vertical curve at Station 49 + 50 (Fig. 25). Note, also, that of the two calculated braking curves, the lower one shows five brake applications, whereas the upper shows seven applications. This difference was caused by the change in grade shown at the foot of the hill from 9 to 10 and 13%, and the shortening of the vertical curve from 200 ft. to 160 ft.

Time is also a factor in the operation. For instance, on the vertical curve from the 11% to the 45% grade at Station 65, 14.3 sec. are required with a line speed of 840 ft. per min. During this time the uncounterbalanced load increases 26 550 lb., or 1 856 lb. per sec., and the strain on the load end of the rope increases 27 090 lb., or 1 894 lb. per sec. If the vertical curve were only 100 ft. long these changes would take place in one-half the time. This illustrates the advisability of long vertical curves.

To keep the engine man advised at all times as to the exact position of the cars the profile and brake-power curve is before him on a vertical cylinder with an indicator wire.

Experience has developed the fact that the construction of the rope used is very important. Various kinds of track rollers and side rollers have been made to protect the line on vertical and horizontal curves, and it has proved economical to use well constructed roller-bearing journals.

A tower is constructed at the head of the incline between the hook-on point and the "snubber". The cars, in switching, pass under this tower while the ropes pass over the top. Beginning at the upper hook-on point and following through to the lower hook-on point, the cable passes over six 20-in. tower sheaves set on a uniform radius of 30.46 ft.; thence it passes over the top of the tower to the 5½-ft. gypsy around which there are 3½ turns, thence over six 20-in. sheaves on the tower set on a radius of 33.84 ft.; thence down over the numerous track rollers to the foot of the hill. These changes in direction of the rope in running its course are necessary to the operation, and rope manufacturers are bending every effort to evolve a product to satisfy the variable conditions under which these ropes must work. The ropes used on a counterbalance are from 1½ in. to 1¾ in. in diameter.

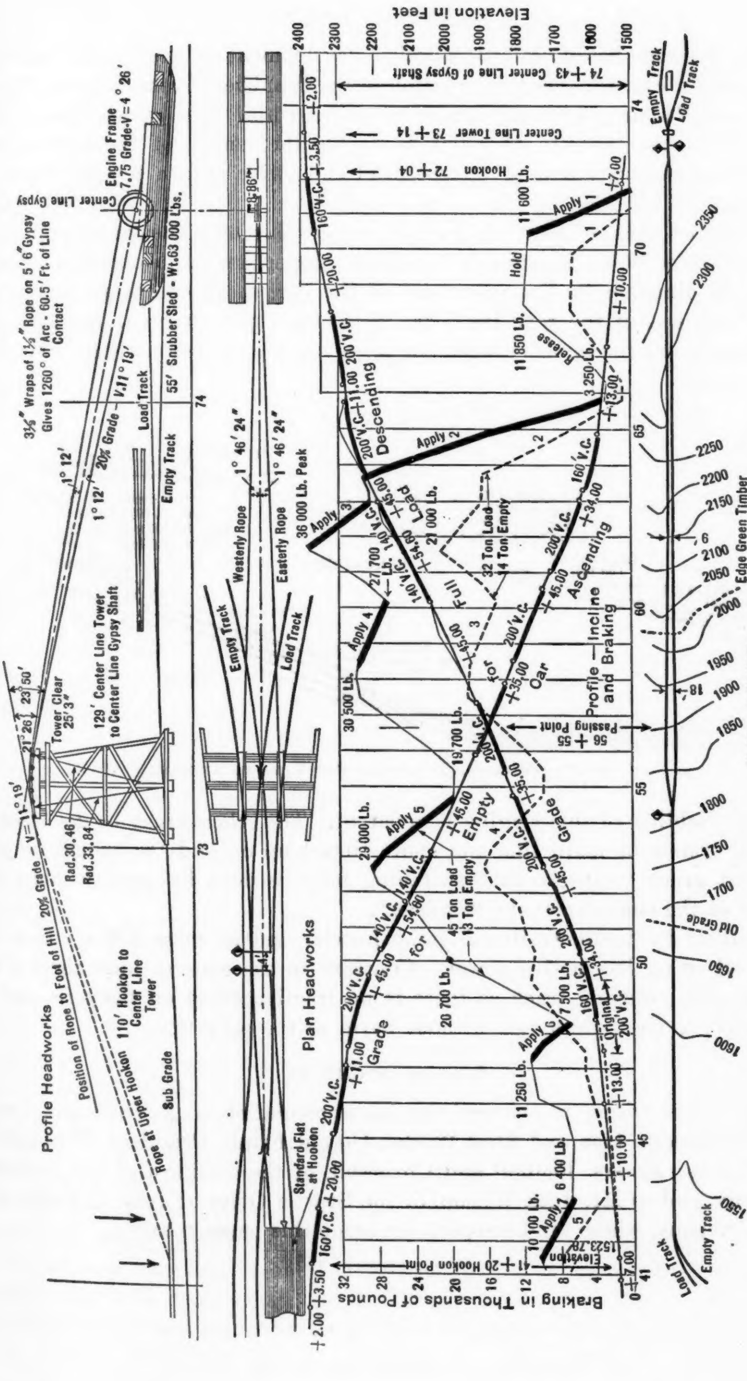

FIG. 25.—DETAILS AND OPERATION OF TYPICAL COUNTERBALANCED LOGGING INCLINE.

The Sessoms Incline

Mr. Sessoms has described* another development of the incline principle, as shown on Fig. 26. The construction of the "snubber" is the same as that for the counterbalance, with the exception of the drum. The gypsy is supplanted by a drum the capacity of which is double the length of the incline, with enough extra for "tail hold" at the dead end swivel and a few wraps on the drum.

The pilot car, as shown in Fig. 26, carries one 6-ft. sheave instead of the original three 36-in. sheaves in the block proper, and two outrigger sheaves 19 in. in diameter at the upper end of the car, so placed that the live and dead line runs of the rope lay 7 ft. $1\frac{1}{8}$ in. from the center of the car. The two sheaves on the bunk between the large block and the outrigger have been eliminated.

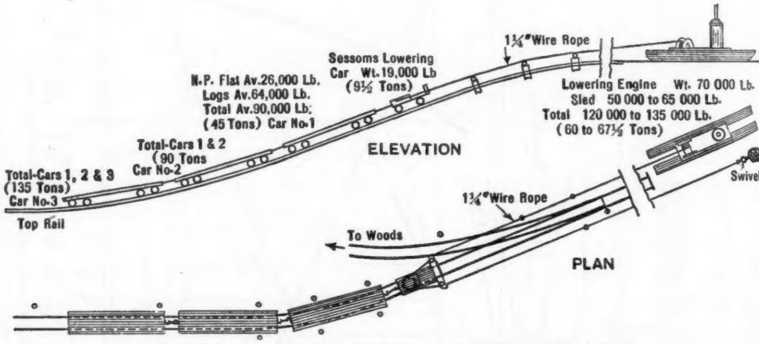

Fig. 26.—Sessoms Lowering Car Type of Incline.

The pliability of this rigging is apparent. The scheme may be described as a track system of main line and spurs tipped up on end far enough to give sufficient grade to insure that switching may be done by gravity under the control of the engineer at the "snubber".

Grading for incline railroads is no heavier and no more difficult than for any road using locomotive power. Contrary to the general idea, there is no trouble with creeping track as there is no tractive effort on the rails, and it is possible to use lighter rail without harm to the operation.

Acknowledgments

The writer wishes to acknowledge his appreciation to A. G. Labbe, President, Willamette Iron and Steel Works; Gerald Frink, President, Washington Iron Works; Messrs. Fennell and Chriswell, of the Pacific Car and Foundry Company, and to Mr. Jack Wilson, of the Seattle Office of John A. Roebling's Sons Company, whose kind co-operation made this paper possible.

* *The Timberman*, 1913.

OCEAN LOG RAFTS

By W. T. Evenson,* Esq.

The subject of "Lumber Rafts" is so broad that this paper has been limited to "Ocean Log Rafts". The Davis raft and the Benson raft are the only two types of ocean-going rafts now in practical use. The Davis raft is built by various loggers and operators under patents held by Captain G. T. Davis. The Benson raft is now used only by the Benson Timber Company, and the Benson Lumber Company, affiliated corporations.

The Davis Raft

The Davis raft is built without the aid of a cradle, and as constructed by the Multnomah Lumber and Box Company, of Portland, Ore., contains from 600 000 to 900 000 ft. of logs. It is much in demand for short trips across rough water, and because of the economy of building. It is practical for use in salt water because it requires no cradle for its construction as does the Benson raft.

The Davis raft is built by first laying a floor of long logs, held together by interlacing cables over and under the logs of the floor which are properly fastened to keep the floor intact until the raft is broken up at its destination. When the floor is completed, logs are rolled upon it, submerging it gradually. At intervals wires are run across the raft to strengthen and bind it together. When the floor is submerged to the proper depth, the cables are fastened across the top and the raft is ready for towing.

The loss on this type of raft has not been large, and the distance of the tow in no instance has been very far. At present, a new Davis raft is in course of development which will take logs of all lengths, and it is estimated will carry in excess of 3 000 000 ft. of logs on long tows.

The Benson Raft

In 1906, Messrs. S. Benson and O. J. Evenson started in the ocean-rafting business at Wallace Slough, Ore., on the Columbia River, engaging Mr. J. A. Festabend to superintend the construction of a cradle and raft. Previous to this time Mr. Festabend had constructed rafts for Captain Robertson, the originator of the ocean log raft. Messrs. Festabend and Evenson constructed a simple cradle with an improved center-locking device, and improved on the towing gear and system used by Captain Robertson.

The first raft built was small as compared with present-day rafts and contained not only piling and saw-logs of all sizes and lengths, but several hundred thousand feet of sawn timbers and lumber for a complete saw-mill which was to be constructed in San Diego, Calif. The towing distance was 1 100 miles and was twice that ever before attempted with an ocean raft. The

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safe arrival and excellent condition of the raft caused considerable comment in San Diego. These rafts produce from 4 000 000 to 5 000 000 ft. of lumber. Of these rafts, seventy-two have been landed safely in San Diego Harbor, only two meeting with misfortune. More than 50% of these rafts have carried deck loads. Cedar poles, spars, shingles, lath, and fence posts have been successfully "decked-loaded" and delivered in good order.

The towing season is from June 15 to September 15. The average towing time is about 15 days from bar to bar. For safety, the rafts are equipped with two lights which will burn for 21 days without any attention from the tugboat.

The Benson raft, which is cigar-shaped (Fig. 27), is built in a floating cradle, or form, which is constructed in sections so that on the completion of the raft the sections can be removed from one side and the raft "kicked out" of the other half of the cradle which is moored to piling. When the completed raft is launched the sections are towed back into place, the center locks are set, and the cradle is ready for the next raft. The Benson raft is not feasible in bacteria infested waters because the cradle will not last more than two years. On the side of the cradle moored to the piling, a derrick moves back and forth on a running line, pushing and pulling the river raft with it as the logs are placed throughout the raft, course upon course, over the length of the cradle. All sizes and lengths of logs are used, but the strength of the raft is dependent on a large portion of the tree-length material. The long logs give the necessary strength to resist the action of the ocean waves and ground-swells.

The cradle keeps sinking in the water as the logs are loaded, and when one-half the raft is completed, a 2½-in. anchor chain is run along the center from end to end. This chain is the back-bone of the raft. Herring-bone chains shackled to the center chain are attached to the five chains that circle the raft at each end. A tow chain, 180 ft. in length, is attached under the raft to the third circle chain from each end, which furnishes an emergency tow chain to be used if the one in service becomes unshackled or breaks from any cause. The raft tows equally well from either end. When pull is exerted the 180 ft. of chain acts as an equalizer, taking care of the surges of the tug, the pull being transmitted by the tow chain through the herring-bone chains to the circle chains on the opposite end of the raft, and the slack created is taken up by the working of the raft in the sea which always tends to lengthen the raft. After the tow chain is in place the process of piling on logs continues until a draft of 26 to 28 ft. is reached. Then the circle chains are fastened around the raft and cinched with the aid of a donkey engine and a set of six sheave-blocks and grab-shackles. The circle chains are of 1⅞-in. anchor chain and are placed at 12-ft. intervals. The total weight of the chain on one of these rafts is about 175 tons. The raft dimensions are 55 ft. wide, about 35 ft. deep, and 835 ft. long, with a draft of 26 to 28 ft.

The great advantage of the Benson type of raft is that it is self-tightening, and there is no tendency to loosen the mass. When the raft is being towed and, in fact, as soon as it is out of the cradle, it tends to flatten and tighten the circle chains, and as it is towed it tends to lengthen and tighten itself.

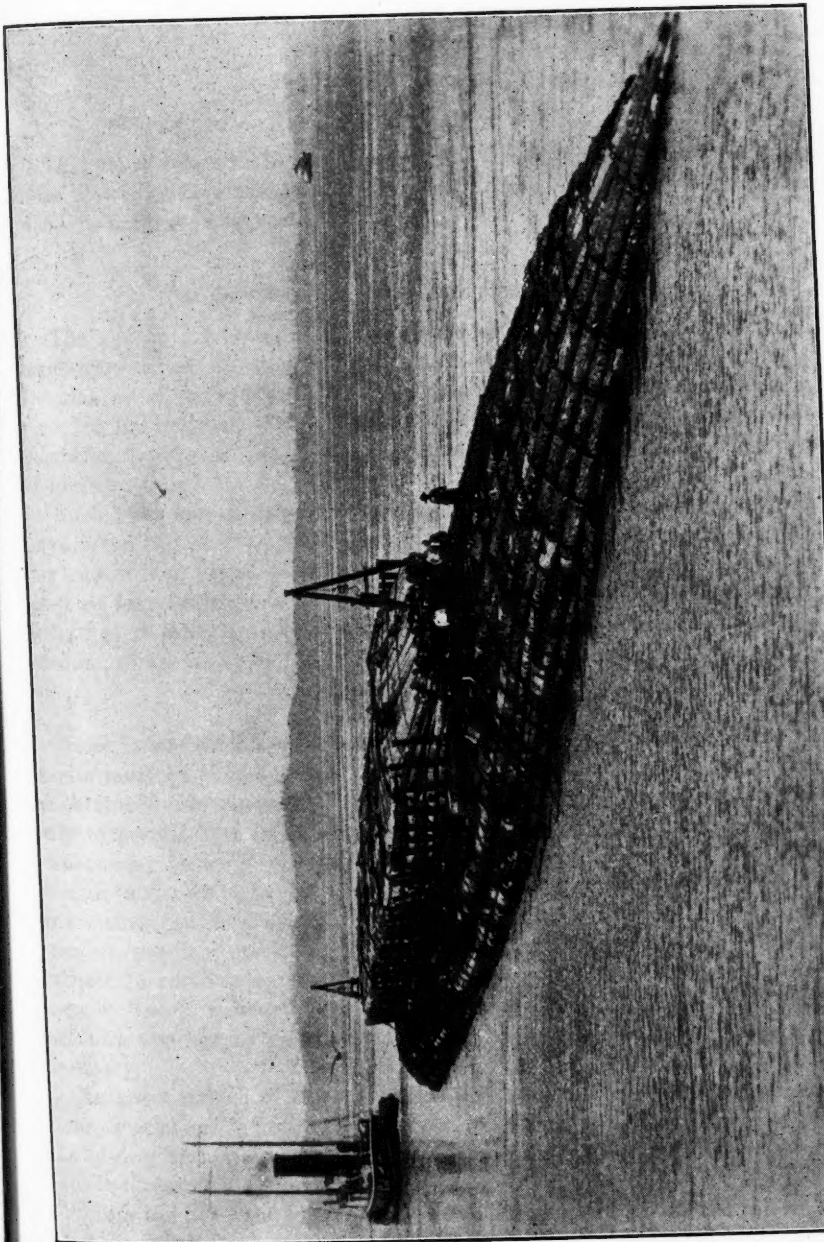

FIG. 27.—VIEW OF LOGGING RAFT.

THE ENGINEERING ASPECTS OF SAW-MILL CONSTRUCTION AND OPERATION

By Bror L. Grondal,* Esq.

This paper refers mainly to saw-mills of the Douglas fir region, and to saw-mills that may be classed as of fair size, for it is in these that engineering problems become more real.

Douglas Fir Logs Are Long and Large

The logs in the Douglas fir region are often very large and heavy. Many reach very great dimensions, both in diameter and in length. The tallest Douglas fir on record was 380 ft. high—which is also the maximum height recorded for redwood. The largest tree ever found had a diameter, at breast height, of 15 ft., and individual trees have yielded as much as 60 000 board-ft. of lumber.

Such trees are of course exceptional specimens, but this will serve to emphasize the fact that Douglas fir is often a very large tree. It is customary to cut logs in long lengths—usually about 40 ft.—to build full flat-car loads, and as these logs often scale as much as 5 000 ft., b. m., with a consequent dead weight of 16 tons, or more, the necessity for a log pond, in which the logs may be dumped from the cars and sorted, becomes apparent.

Log Ponds Offer Problems

Here is one of the engineering problems that confronts the mill designer. A site must be chosen where level land for the mill itself and the yard may be obtained. Moreover, this site must have good transportation facilities, not only to permit logs to be hauled to the mill, but in order that the finished lumber may be shipped; and more, this site must be where a log pond large enough to store a sufficient number of logs to run the mill at least three weeks or a month can be provided. A fairly large Douglas fir mill, cutting 300 000 board-ft. per day, operating on a double shift, will require, therefore, a pond at least 15 acres in extent, preferably larger. This pond must be reasonably deep, so that it will not fill up quickly with dêbris. A good water supply, to replenish the loss by evaporation and to keep the pond from fouling, is also necessary.

No great stretch of the imagination is required, therefore, to assume that dams must often be built, reliable dams, but cheap, for the margin of profit in the sawing of logs is very small, and the overhead cost of the mill must be held to a reasonable figure.

From the pond the logs may be lifted into the mill with the aid of a sling hoist, by which the logs are parbuckled to the deck; or they may be carried up on an endless chain, known as a log jacker, log haul, or log chain.

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Heavy Logs Demand Power Devices for Handling

As the log deck, which must be heavily built, is inclined toward the carriage, a log-stop must be provided to prevent the log from rolling forward when not wanted. This, as well as a log kicker, to kick the logs out of the haul-up, when such a device is used to bring the logs into the mill, must be actuated by a steam cylinder. A log turner, to load the logs on the carriage, and to turn them as occasion demands during the sawing, must also be provided. A "Simondson" turner is usually provided for larger logs, a "nigger" for smaller logs, and an overhead canting gear for emergencies and to handle extremely large logs.

Automatic Log Carriage Equipment Saves Sawing Time

The carriage must be large enough, and sufficiently strong, to accommodate the largest logs that are likely to be sawed. The carriage is usually operated by a reversible twin steam engine geared to a drum, around which is wrapped a cable. This passes through sheaves at both ends of the track, and both ends are attached to the carriage. In one of the most modern mills, an electrically driven motor carriage feed is provided. In still another modern mill a huge steam "shot-gun" feed has been installed. This feed, consisting of a 48-ft. steam cylinder, piston, and piston rod, dwarfs the familiar "shot-guns" of mills in other regions. On the carriage itself are skids of steel or "blocks" on which the log rests. On these skids are also "knees" against which the logs are held by power or hand-operated "dogs". Two types of quick-acting, automatic, power, dogging devices have been developed—one a compressed air apparatus, and the other an electrically controlled mechanism. Both types are good and save valuable time.

Set Works Are Electrically Operated

The log is moved out into the line of cut with the aid of the "set works", which is now usually electrically driven, and which moves the knees forward on the blocks an equal distance, when the log has been squared, but which must also be built in such a manner that one knee can be moved at a faster rate than the others on the carriage.

Taper of Giant Logs Complicates Sawing Methods

Here, again, is an engineering problem in operation. Although many logs are approximately round, all of them taper. The most valuable material in the log is in the outer part, which is free from knots. If the taper of the log is disregarded in sawing, expensive waste occurs. The scope of this paper does not permit extended discussion of the proper sawing of logs, and the so-called taper-sawing, but engineers who are interested in mathematics as an abstract science will find some very interesting problems in it. A "head sawyer" can almost make or break a mill, and it is obvious that he must be, intuitively, a good engineer. It may be of interest that one of the best head sawyers the writer has ever known was a graduate civil engineer who liked his "job".

Band-saws are now almost exclusively used in Douglas fir mills; for large logs, the single-cutting type, and, for smaller logs, the double-cutting type. Band-saws produce less saw-dust than circular saws and have other marked superiorities in producing a high grade of lumber. Band head-saws with 10-ft wheels are very common, and larger saws are in use in many plants.

Head Saws Do not Govern Mill Capacity

In other regions most of the sawing is done on the head saw, and, in the Baltic pine region of Europe, gang head saws are commonly used. In Douglas fir mills the head saws are used primarily to break the logs into large cants, and to square up timbers from the interior of the log. A mill of comparatively small capacity may be provided with just as complete deck equipment and as large a head saw as one of large capacity. The capacity of a mill depends to a great extent on the type of equipment after the head saw has been passed. Two head saws, one single cutting and one double cutting, represent the "head end" of one of the largest mills ever built.

A band head saw of the type commonly used demands a motor of approximately 400 h.p., and larger motors have been used. These motors usually drive through belts, although at least one machinery manufacturer is now successfully building such a saw with a direct-connected motor suspended from the massive extended base of the machine. Roller and ball-bearings, which reduce the starting demand and the average input of power materially, are now often used in mounting the wheels of band mills.

As the cants drop from the carriage, they are carried away on "live rolls", of steel, and are picked off by means of "jump chains" to the chains on which they are conveyed to the edger-feed table. Mechanical liners are used for lining up the cants before they go through the edger, for they are often too heavy to be moved by hand.

Heavy Edgers Require Mechanical Saw Shifters

The edgers are large, to accommodate wide and thick cants—often as large as 14 by 72 in.—and are usually provided with mechanical saw shifters to move the large and heavy saws. The largest edgers require as much as 400 h.p., and are often equipped with direct-connected motors. At the rear of the edger is a series of live rolls carried on a truss extending over the chains which carry the edgings and slabs to the "slasher". The boards coming from the edger are dumped over the end of these rolls on chains leading to the automatic trimmer.

Edgers in Douglas fir mills often serve a dual purpose. Heavy boards and cants are edged, to free them from bark and wane. In this respect, the edger is operated in the manner that its name implies it should be. On the other hand, the edger is commonly used to re-saw cants into thick boards, which, in turn, are re-sawed later. In this process of re-sawing, the heavy kerf of the circular saws used in the edger causes a great waste of valuable wood, for the teeth of the saw are often swaged to a width of almost $\frac{1}{3}$ in.

Timbers cut on the head saw go out to the timber dock on live rolls, passing right through the mill. Slabs are kicked off by means of jump chains to the

slasher chains below. Edgings and slabs pass under a battery of ten saws, spaced 4 ft. apart, all on the same arbor, driven by a direct-connected motor of approximately 50 h.p.

Mechanical Automatic Trimmers an Innovation

The boards that have passed through the edger go to the trimmer which, in all modern mills, is of the automatic type, with twenty-one saws that can be dropped at the will of the trimmerman so that the boards passing under the saw on chains can be trimmed to proper length and in such a manner that a proper grade of lumber will be produced. Three types of automatic trimmers are in use. In the most common type the saws are lifted by compressed air acting on pistons in cylinders. In extremely cold weather, the air lines may become frozen, but this can be prevented by introducing alcohol into them. On another type the saws are lifted by means of electric solenoids. Several modern mills have such installations. A new and vastly improved type is purely mechanical and as it is free from complications it bids fair to become the standard type of trimmer. In every case, the trimmerman operates the saws by remote control from a cage facing the bank of saws, a keyboard being provided for his convenience. The saws in the trimmer are belted to a series of pulleys on a single shaft, driven by a direct-connected motor of about 50 h.p. After the lumber passes through the trimmer, it is discharged on a sorting table, from which it is pulled for re-sawing, re-edging, or re-trimming.

Pony and Gang Saws Simplify Cutting of Vertical Grain Material

As has been indicated previously, the capacity of a mill in this region depends on the amount and nature of machinery after the lumber has passed the head saw. So-called "pony rigs", for re-sawing cants produced on the head saw, may be included in the design of the mill. A pony rig must have a cant deck with chains, turners (usually of the "nigger" type), and a carriage similar to that used at the head rig. Means must be provided for conveying the cants from the head rig to the pony rig, and, therefore, the pony rig is usually the first piece of re-sawing equipment in the mill. As the pony rig usually consists of a band-saw cutting a narrow kerf, its use is advantageous.

Another machine, which is achieving deserved popularity in the mills of this region, is the vertical reciprocating gang saw. These saws are often built to accommodate cants 12 in. thick, and sometimes have a width of 4 ft. They require extremely heavy foundations, as the vibration caused by the reciprocating parts is very great, although, recently, so-called "balanced" gangs have been introduced. The gang saw is always belt-driven, and requires, in the larger sizes, a 100-h.p. motor.

The pony head rig and the vertical gang saw save much lumber and greatly simplify the cutting of vertical grain material. They also greatly increase the cost of mill construction and complicate the work of the mill designer, but are almost indispensable in large mills.

In the re-sawing room of the mill, the layout must be such that lumber may be pulled from the sorting chains for re-sawing, re-edging, or re-trimming,

and returned to the same chains in such a manner that it may be re-worked over and over again. At the same time, sufficient storage room must be provided at each machine, so that the mill can be kept running at the highest efficiency in every department.

A vertical gang saw will usually produce enough lumber in 1 or 2-in. thicknesses to keep a trimmer busy. Plenty of storage space for cants must be provided in front of the gang, so that it may be kept supplied with proper material.

Two types of band re-saws are commonly used. The most popular is the vertical roller band mill with 6, 7, or 8-ft. wheels. In this device the boards to be re-sawed are fed to the saw on edge between rollers, and, therefore, the stock to be re-sawed must have at least one straight edge. A horizontal band re-saw, in which the lumber is re-sawed while flat, does not demand square-edged stock, and, therefore, has advantages that are quite apparent. On the other hand, horizontal band re-saws require more skillful filing, because if the saws are not properly fitted, very poor lumber may be produced. Horizontal band re-saws are sometimes used for re-sawing slabs, and are useful for this purpose.

After the lumber leaves the re-sawing room it passes out on the sorting chains, where it is graded and sorted, being pulled from the chains by hand and placed on trucks.

Various types of automatic drop sorters have been installed in the largest Douglas fir mills, but these are expensive and are not as practical in this as in other regions, for the multiplicity of sizes, grades, and length produced greatly complicate the problem of automatic sorting.

Kiln-Drying of Entire Output of Mill a Recent Development

A recent development, and a logical one, has been the kiln-drying of the complete output of the mill, with the exception of timbers. Air-drying on the Pacific Coast is very slow and expensive, for the relative humidity is comparatively high, except during the summer months. The cost of shipping green lumber is enormous, for almost one-half its weight is due to moisture. All this water cannot, of course, be removed, but a large proportion can be eliminated by proper kiln-drying. As the freight charges paid to the railroad companies sometimes exceed the value of the lumber at the mill, the desirability of kiln-drying becomes apparent.

Edge stacking for the dry kilns is common practice at most of the larger mills, and is very desirable, as it reduces the cost of stacking and unstacking, increases the capacity of the mills, and permits better circulation within the kilns. In dry kilns of modern design the circulation of the heated air is produced mechanically by fans, and the relative humidity and temperature are held at predetermined points with the aid of automatic controlling and recording instruments.

Use of Electric Drives Removes Old Restrictions in Mill Design

A few years ago, when all saw-mills were driven by mechanically distributed power, the saw-mill designer was so burdened and harassed by mechan-

ical details that mill-operating efficiency was often sacrificed to make possible some particular drive. To-day, these restrictions have largely disappeared, due to advances that have been made in electrical engineering. All the larger mills are electrically driven, and complicated shafting and counter-shafting, huge bevel-gears, "muley drives", and other power-stealing equipment have been relegated to the scrap heap.

To-day, the proper order of design is first a detailed consideration of the floor plan of the mill; second, the building; and, finally, the drives. The first electrical installations made use of the so-called "unit drive" system, by which power from more or less centrally located motors was distributed to various parts of the mill by belting and shafting. Now, however, the mill designer can choose between a number of types of highly efficient self-contained speed reducers, which can be coupled directly to the motors, and, therefore, most of the belting and shafting in the saw-mill have disappeared.

High-grade, steel roller chain has greatly decreased the loss due to breakdowns. When the crew of a saw-mill must stand idle for a few minutes, the loss to the mill owners quickly amounts to an astounding figure.

Timber-Frame Mill Buildings Offer Advantages

Saw-mill buildings are invariably of Douglas fir. One steel and concrete mill has been erected, but the superiority of Douglas fir as a building material when maximum stiffness and strength at minimum cost is required, is apparent to all saw-mill owners who have inspected this building. Buildings must be planned to conform to the proper sawing-floor requirements. Large roof trusses can easily be built from Douglas fir. Some idea of the type of timbers that can be cut from fir trees is illustrated by the huge timber at the Forestry Museum of the University of Washington. This timber is free from wane and big knots, yet it is 18 by 18 in. in section by 156½ ft. long.

In planning the sawing floor, attention must be paid to the waste conveyors, for débris must be conveyed from the mill to the burner as rapidly and efficiently as possible. The proper design of a burner that will not throw cinders is an engineering problem that has not yet been solved.

Summary

It should be emphasized that the design of saw-mills in the Douglas fir region calls for the highest kind of engineering skill. Huge power plants must be built, the mill must handle enormous logs in an efficient manner, as well as small logs, for the trees taper, and small logs from the upper part of the stem constitute a fair proportion of the daily cut. Hemlock and cedar mills differ markedly from Douglas fir mills, because in such mills shorter logs—although they may be large in diameter—must be cut.

Every log that comes into the mill is a new problem—no two are exactly alike. A new engineering problem arises every minute, and the ingenuity of the expert mill designer is often sorely taxed—yet nowhere in the world are saw-mills as relatively efficient as in the Douglas fir region.

ELECTRIFICATION OF LOGGING AND MILL EQUIPMENT

By L. D. Beach,* Esq.

The writer has endeavored to confine this paper to a brief yet comprehensive description, of the electrical installation in general and the equipment operated by The Long-Bell Lumber Company at Ryderwood, Wash. Space does not permit mention and description of a great number of details applying to both the transmission system and the various units of electric logging equipment.

The application of electricity for driving lumber mills is not new. A number of mills were wholly, or in part, electrically driven prior to the application of electric power to logging machines. Manufacturers of electrical equipment and others have quite thoroughly covered the subject on the application of electric drive in lumber mills; however, the writer believes that it would be of interest to outline briefly several important features in electric mill drives that have not heretofore been general in their application. Comparatively recent developments in the design of motors indicates that in selecting equipment for new mills, or for mills undergoing extensive changes from steam to electric drive, serious consideration should be given the motor installations.

Power Plant and Transmission Features

Power for the operations at Ryderwood comes from a power plant located on the Company's mill site at Longview, Wash. Wood refuse and hogged fuel from milling operations is used exclusively at the power plant. The present power plant has a capacity of 18 000 kw. and consists of three 6-000-kw. turbogenerators and eight sterling water-tube boilers of 1 200 h.p. each.

Current is generated at 13 200 volts and is transmitted over three-conductor underground cables to a sub-station at the mill site, approximately 1 mile from the power plant. This sub-station serves the City of Longview, Wash., and supplies energy at its southern terminus to the high-voltage transmission line to Ryderwood.

At the sub-station the current is stepped up to a transmission line pressure of 66 000 volts, through a delta-connected outdoor bank of three 2 000-kv-a., single-phase transformers, these being connected to the transmission line through an automatic, oil circuit breaker.

The total length of the transmission line from the sub-station at Longview to the sub-station in the woods (approximately 2 miles beyond Ryderwood) is 32 miles. This line is of No. 2/0, seven-strand, bare copper cable, strung on 50-ft. cedar poles, with pin insulators on the straightaway, corners and curves being made on suspension disk insulators.

* Chf. Elec. Engr., The Long-Bell Lumber Co., Longview, Wash.

The sub-station at Ryderwood is of the outdoor type and contains a bank of three delta-connected, 1 500-kv-a., single-phase transformers, which reduce the potential from 66 000 volts to 13 200 volts.

The secondary bus is arranged with five positions for the outgoing 13 200-volt feeders, each of these feeder positions being controlled by automatic oil circuit breakers of the three-tank type and the necessary disconnecting switches, choke coil, lightning arresters, etc. The feeder lines leaving the Ryderwood Sub-Station radiate through the woods to portable sub-stations, where the line potential is stepped down to the motor operating voltage of 600. Instead of building the lines in the woods parallel to the main line and spurs of the railroad, they are on lines dividing the yarding areas. This practice of routing the overhead lines saves from 25 to 35% of that which would obtain if the overhead lines were constructed parallel to the main logging railroad and spurs.

No poles for line construction are set in the woods, at corners, and in open spaces, the lines being carried on single disk suspension insulators fastened to the trees by short lengths of steel chain or steel cable, which are fastened in turn by bolts or cable clips.

A large portion of the woods lines are temporary, and the fastening of the lines and insulators close to the tree trunk requires only the occasional use of cross-arms. When a portable sub-station is moved to a new spar-tree location, the line feeding it is extended or moved as the case requires.

There are twelve portable sub-stations, each consisting of a 600-kv-a., 3-phase, low-reactance transformer, which steps down the potential of the woods lines from 13 200 to 600 volts. These portable sub-stations are mounted on a pair of log skids, approximately 14 ft. long, with six heavy timber cross-members connecting the two skids (Fig. 28), the entire structure being held together by heavy bolts.

The upper frame of the transformer sled is composed of 6 by 6-in. timbers, this framing also carrying the necessary auxiliary apparatus, such as the circuit breaker for the incoming line, choke coils, lightning arresters, etc.

The secondary leads from each of the portable sub-station transformers run into a steel enclosing cabinet containing copper connecting bars, arranged so that one, or if required, two, of the portable three-conductor, 600-volt, wire-armored cables can be taken from each sub-station for connection with the electric logging units. The steel cabinet also contains fuses, meters, and other equipment for metering the energy consumed and switching to one, or possibly two, logging machines which it may be supplying.

In the portable sub-stations, consideration was given to the necessity of moving them each time a unit of logging equipment was moved from one spar-tree location to another. The sub-stations are moved on railroad flat-cars, the loading and unloading being accomplished by the use of a railroad steam crane. Their construction is, therefore, unusually rugged so as to withstand rough handling.

The cable which connects the logging machines electrically to the portable sub-stations consists of three rubber-covered conductors, each of 300 000 cir.

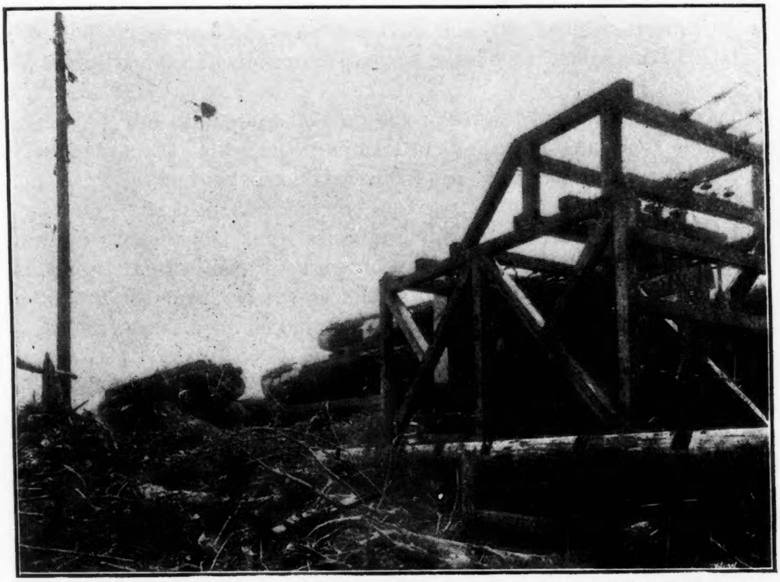

FIG. 28.—SKID-MOUNTED, 600 KV-A., PORTABLE TRANSFORMER SUB-STATION.

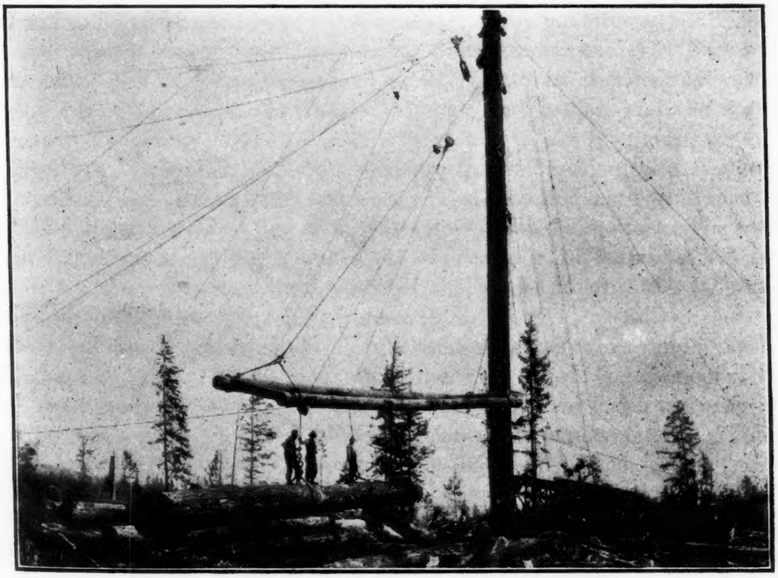

FIG. 29.—COMBINATION YARDER AND LOADER.

mils, laid with tarred jute fillers, and wrapped with four layers of varnished cambric tape and one layer of half-lapped rubber-filled tape. The whole is enclosed in a galvanized steel wire armor wound with a 5-in. pitch. The short pitch in the armor winding and the fact that the cable is devoid of a lead sheath, make it very flexible and easy to handle as compared with standard cable.

In service, the cables are laid on the ground frequently through pools of water, and they are called on to withstand the rough treatment characteristic of such locations and service. The cables are made up in lengths of 500 and 250 ft. each. It is intended that the maximum transmission distance between the portable transformer sub-station and the logging machine should be 1 000 ft. However, the average distance does not exceed 500 ft.

In the woods, no provision is made for handling or moving the cable on reels. When a cable is to be moved from one location to another, it is lifted on to flat cars and laid in long loops.

Electrical Logging Equipment

At present, the electric logging equipment operated at Ryderwood consists of ten units, comprising six car-mounted combination yarders and loaders one of which is shown in Fig. 29, two car-mounted interlocking skidders and loaders as shown on Fig. 30, and two single yarders mounted on sleds. This equipment was manufactured and furnished equally by the Washington Iron Works, Seattle, Wash., and the Willamette Iron and Steel Works, Portland, Ore., each of these companies supplying three combination yarders and loaders, one combination skidder and loader, and one single sled-mounted yarder. The essential difference between the two makes of machines is that the Willamette units use gearing which is controlled and shifted by compressed air to obtain high and low speeds on the main yarding line, whereas the Washington Iron Works units uses a single two-speed motor to obtain the desired yard-line speeds. Fig. 31 shows a combination skidder and loader, and the spar-tree rigging.

The three Willamette yarders which are in combination with loaders and car-mounted, and one yarder which is sled-mounted, are equipped with 300-h.p., 550-volt, 450 rev. per min., variable speed type motors. The average speeds of the main yarding line are 775 ft. per min. in high and 375 ft. per min. in low gear. The average speeds of the haulback are 1 900 ft. per min. in high and 900 ft. per min. in low gear.

To eliminate mechanical shock in changing gears while under load, the gear change mechanism is such that the high-speed engagement slightly anticipates the low-speed release, the high-speed friction band dragging on its drum for a fraction of a second before the low-speed band releases. The result is an over-lapping of the two speeds which affectually prevents any surge on the yarding line during the gear (and consequent speed) change.

The Willamette loaders are equipped with 200-h.p., 550-volt, 600 rev. per min., variable speed hoisting motors and 35-h.p., 600 rev. per min., variable speed boom swinging motors. The hoisting lines travel at an average rate of 675 ft. per min., and the boom swinging lines at a speed of 390 ft. per min.

The Willamette interlocking skidder has a 450-h.p., 600 rev. per min., variable speed motor which operates the main line at an average speed of 750 ft. per min. in high gear and 360 ft. per min. in low. The haulback line travels at an average of 2 100 ft. per min. in high speed, and 960 ft. in low.

The three combination yarders and loaders are mounted on cars 46 ft. 9 in. long equipped with two four-wheel trucks. The weight of each of these units, devoid of any cable, is approximately 175 000 lb. The interlocking skidder and loader is mounted on a 60-ft. car with truck equipment similar to the others. The weight of this unit, without any cable, is 247 000 lb. All car-mounted units are equipped with hydraulic jacks to raise the car body, thereby relieving the truck springs of weight. Compressed air at 100 lb. pressure is used for the operation of the frictions and signals.

The Washington Iron Works yarder and skidder motors are of the two-speed type, each developing 300 h.p., and 200 h.p., respectively, on low and high speeds. The speed change is accomplished electrically by re-grouping the stator winding of the motor to obtain either 24 or 12 poles; the loader motors are 250-h.p., variable speed type and the boom swinging motors are 50-h.p. of the same type.

In addition to the essential difference in effecting speed changes on the main lines, a further difference between the two makes of machines obtains in the motor controls. The Washington Iron Works units are supplied with electro-pneumatic operated equipment for control of the motors. The contractors are, therefore, directly operated by compressed air, instead of magnetic control. The pilot valves controlling the air cylinder are, in turn, operated by a direct-current control circuit, energy for which is furnished by a single, 1-kw., 35-volt, motor-generator set. As these units require compressed air for the operation of frictions, signals, and also control equipment, each is equipped with two 52-cu. ft. per min., Westinghouse air-cooled compressors.

The electrically operated logging machines described in the foregoing do not comprise the greatest number of units for the ultimate woods operations. The transmission lines, sub-stations, and equipment are of a design and size to supply power to at least ten additional units, each of a size not less than those now in operation.

The equipment at Ryderwood is to date the largest single installation in existence—the present and ultimate requirements were known well in advance, and the entire project was planned and built accordingly. In no instance has there developed a failure of the electric distribution system or any of the machine units to perform the maximum duty expected. In fact, every phase of the logging development has more than fully met the expectations of its designers and the company operating it. The total number of motors involved is 42, and the total aggregate horse-power, 5 750.

Table 1 shows the consumption of electricity, at point of use, for all the equipment operated at Ryderwood, with the output, in log-feet, for a 9-month period. Table 2 covers the performance data applying to each machine unit. The electric energy consumed by each unit is also included. These data cover

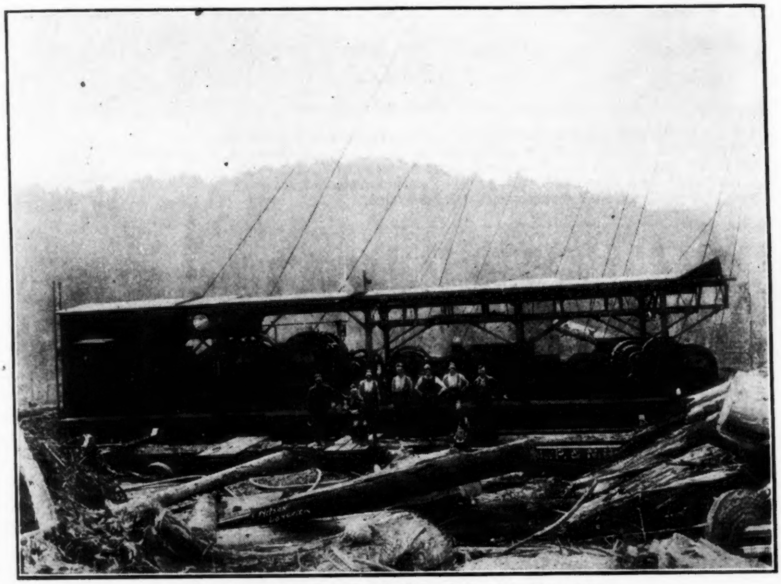

FIG. 30.—COMBINATION SKIDDER AND LOADER.

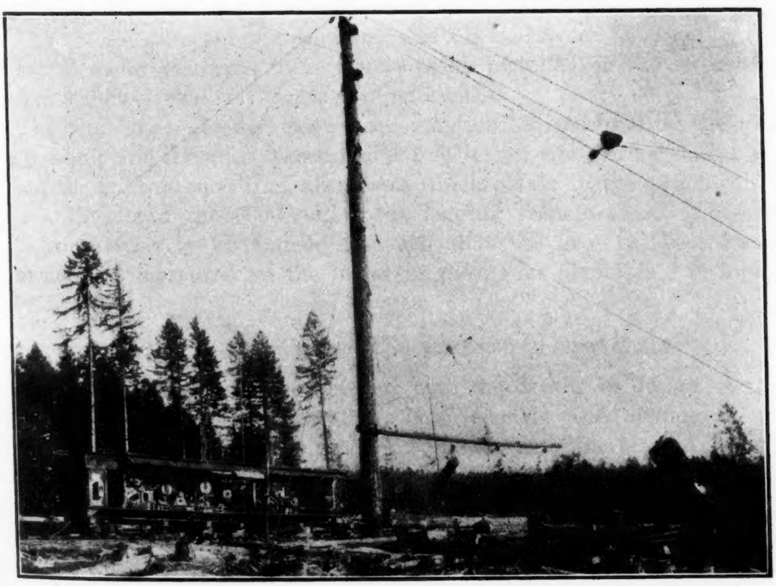

FIG. 31.—COMBINATION SKIDDER AND LOADER, SHOWING SPAR-TREE RIGGING.

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two periods of operation, one of nine months from April to December, inclusive, 1925; and one period of five months from January to May, inclusive, 1926.

TABLE 1.

Month, 1925.	Equipment Consisting of Six Combination Yarders and Loaders. Two Interlocking Skidders Equipped with Loaders, and Two Sled-Mounted Yarders Which Always Work in Conjunction With Either a Combination Yarder and Loader, or an Interlocking Skidder Equipped with Loader.		
	Kilowatt-hours.	Net commercial scale, in log-feet.	Average log-foot per kilowatt-hour.
April	122 300	14 141 514	115.6
May	150 700	17 549 712	116.4
June	153 100	20 827 225	136.0
July	138 600	19 128 742	138·0
August	179 500	22 609 813	125.9
September	181 300	22 416 499	123.6
October	178 600	23 952 439	134.1
November	167 900	21 414 356	127.5
December	128 000	16 877 645	131.8
Total	1 400 000	178 917 945	127.9

Summary:	Electric Energy.
Total kilowatt-hours	1 400 000
Total log-feet (commercial scale)	178 917 945
General average, 9-month period	7.8 kw-hr. per 1 000 log-ft.

The net electric energy consumption of 7.8 kw-hr. per 1 000 log-ft. is based on actual meter readings, these meters being installed on the secondary side of each 600-kv-a., 600-volt logging transformer.

The total gross electric energy consumption as measured by the meter in the power plant includes transmission line losses, transformer losses at each end of the transmission line, also losses which obtain in the 13 200-volt woods branch lines and the 600-kv-a. woods logging transformers. These losses, it is conservatively estimated, are such that the gross kilowatt-hour consumption (as measured by the meter in the power plant) is 9.15 kw-hr. per 1 000 log-ft.

Motors and Control Equipment for Saw-Mills

It is to be noted by those familiar with the details in design of saw-mill equipment, that without exception, both electric and steam-driven mills include in their equipment a number of one-way rotation, also reversing power transmitting, devices commonly referred to as "spur friction drives", a number of which are used in the drive to live rolls.

These transmissions are inefficient, their performance is not positive, and their operation is expensive for maintenance and renewals. It is possibly the belief of some mill engineers and operators that such drives are essential, therefore a necessary evil. However, certain types of electric motors are available which, with their proper application, together with suitable reduc-

tion gears, would successfully replace any drive heretofore transmitted by one of the spur friction type.

These motors with push-button operated one-way rotation, or reversing, control equipment, as the case may require, together with suitable reduction gears, are capable of performing fully as many starts, stops, and reversals per minute as have ever been required through the medium of spur-friction drives. It is, therefore, possible, and in every respect practical, to equip an electrically driven mill eliminating all such drives.

TABLE 2.

NINE-MONTH PERIOD, APRIL TO DECEMBER, INCLUSIVE, 1925.

Machine.	Total log-feet for each machine.	Average hourly operation (log-feet per hour.)	Maximum recorded daily output of 8 hours, per single unit, in log-feet.
A	13 116 810	11 278	153 106
B	21 122 468	14 796	243 418
C	21 042 311	13 739	259 486
D	22 060 574	14 667	259 365
E	25 245 879	16 713	267 611
F	24 506 732	16 497	*344 945
G	27 235 955	19 384	271 008
H	24 587 216	16 085	240 250
Total	178 917 945	......	....

General Average (per Single Unit):
(Based on total output and total net hours in service), 9-month period, in log-feet per hour.....15 381

FIVE-MONTH PERIOD, JANUARY TO MAY, INCLUSIVE, 1926.

Machine.	Number of logs.	Log-feet. (commercial scale).
A	8 809	9 804 835
B	13 030	12 428 282
C	13 456	10 787 578
D	12 284	10 490 658
E	9 895	15 755 568
F	9 984	15 236 927
G	12 098	18 097 727
H	6 797	9 701 341
Total	86 353	102 302 916

* Machine F, 344 945 log-ft. maximum for year 1925. This machine logged 443 552 log-ft., or 36 cars on March 31, 1926, which constitutes an average date, and is the highest output for one day attained by a single unit.

In the design of electrically driven mills, regardless of their size, the engineer in selecting induction motors for driving various units of equipment, has always been confronted with the problem of providing motors of adequate size to meet any reasonable operating condition.

Furthermore, occasional operating conditions would arise in which the maximum load on the motor would be in excess of what it could be expected

to perform for any great length of time, and this resulted in decreased production.

If the engineer provided motors of adequate size to insure meeting occasional maximum loads, the result was that the motor operated underloaded about 85% of the time, this underloading being conducive to a low power factor.

Electrical manufacturers have made great improvements in synchronous motors and control equipment and their cost has been materially reduced. Both have been simplified and there is now available push-button operated control for them. They therefore do not require skilled attendance for their operation, and might be classified as general purpose synchronous motors, inasmuch as their starting torque is fully as great as that of the squirrel-cage motor, their starting current corresponding to full load starting torque does not exceed 450% rated full load current, and their pull out, or maximum running torque, under normal voltage and frequency, is not less than 175% of the rated full load torque.

In the design of new mills, or in making extensive alterations in existing mills, consideration should be given to the application of synchronous motors for driving fans, compressors, positive pressure blowers, trimmers, slashers, and, under certain conditions, edgers. The correct application and use of such motors would permit, where required, and remedy the evil attendant on, the use of induction motors of comparatively large size to meet maximum load conditions. The results would be higher efficiency and increased production.

ECONOMIC ASPECTS OF REFORESTATION

E. T. Allen,* Esq.

The viewpoint of the lumberman, the forester, and the public on reforestation differs widely, or reforestation problems would be nearer solution.

In addition to being as interested as any citizens in the general welfare, lumbermen want to perpetuate their industry and land values. Probably they are no more nor less individually selfish. Their first concern is to stay solvent, furnishing every one lumber at a price that all can afford to pay. They have no inclination or ability to grow future forests at a loss for a public apparently unwilling to share the cost or even to provide encouraging fire and tax conditions. Experience makes them skeptical of public co-operation; tradition confined to conversion and marketing, not production, gives them no independent assurance. The broad advice they receive from foresters is so consistently optimistic that they doubt its reliability, knowing that no business enterprise is automatically secure under the conditions involved. They find it equally inconsistent in matters of important detail for, on these, foresters disagree notoriously; but as never before, lumbermen now face the last virgin supply, consequently the reforestation problem also, and they know it. They are endeavoring to solve it as a business problem. To them its economic aspects mean success or failure.

The typical viewpoint of the forester, with exceptions, is that of a professional man exalted almost into a creed by traditions of his teaching. He feels ethically constrained to seek the ideal in forest-growing, as an engineer wants to achieve it in his designing, and mostly he believes this an end in itself, bound somehow to be justifiable. Forest land not doing its best, like avoidable timber shortage, seems to him an economic crime that must be unprofitable for any one concerned. He would compel the owner to save himself such economic loss, even if he does not go so far, as many do, as to advocate the regulation of lumbering by police power in order to save the community from it. His creed is production to the capacity of the land. Seldom does this creed consider profit and loss, or who shall pay the bills, as other than secondary details that will straighten out somehow through some economic justice that he takes on faith and does not feel it his responsibility to guarantee. He sees the same obstacles the lumberman sees, and often advocates the same public remedies, but they do not assume to him the same economic aspect. He is neither the short-sighted lumberman nor the short-sighted public. He owns no lands, pays no costs, takes no risks, buys or sells no lumber. He is too often a doctor prescribing something analogous to the long rest or the trip to Europe. If the patient cannot take it, the doctor's economic aspect remains the same as ever.

* Forester in Chg., Western Forestry and Conservation Assoc., Portland, Ore.

Viewpoints Must Be Compromised

The essential differences between these viewpoints, creating misunderstandings and mutual suspicion, have done much to retard forestry in America. Perhaps each side has been equally narrow in the past. At present, when lumbermen as a class are at least interested, seeking to learn how far they can go, and how, many actually trying, the writer thinks they are showing more progressiveness of thought than the foresters as a class, who incline more to stand uncompromisingly on the ground they first occupied. When the lumberman can be induced to go as far as he sees the way clear, the forester's duty is to get out of the cloudland of an ideal future and down to the ground of to-day, so they may begin together with combined and common vision, together taking step by step forward to make forestry a self-sustaining but intelligent business that can be conducted, as well as dreamed of, for the public good.

Therefore, in this paper, to be read by professional men, and aware that it may make him an apostate in the eyes of many fellow foresters, the writer is going to express for the first time publicly a conviction that has been growing during his twenty-seven years in the work and is now positive as a result of analyzing the present situation. That is, that very great harm is done by this tradition that forestry is necessarily a profession with professional creeds and ideals never to be compromised.

Medicine quite clearly is, probably law. Engineering may be, although perhaps it sometimes involves the point the writer wishes to make. However, whether or not forestry is a profession, forestry itself as private enterprise—commercially growing and using trees anywhere but in words, or publicly subsidized forests—is a business. It is intelligent lumbering or else lumbering is practical forestry. It is not advising, but doing; with all responsibility for using the best judgment at command under existing conditions and exigencies; always a compromise, always shifting, always seeking to learn and to improve conditions, but not ignoring those confronted. Ideals and ethics, yes; but those of business may be, and usually are, as high and prideworthy as those of a profession. The real measure of each is actual resultant good.

When foresters see this, and become as proud of making the business of forestry both honorable and successful as they are of making the profession of forestry honorable if unsuccessful, and when enough lumbermen see they mean it, the three viewpoints will be reduced to two—their's and that of the public.

Public Often Inconsistent

The viewpoint of the public is simple; it merely wants the primeval forest undisturbed for sentimental reasons, considers the destroyer of a tree a social criminal, and at the same time wants lumber produced by wasteful methods if these make it cheaper. If production is so cheap that the producer goes broke, this is assumed to be a public gain. It wants reforestation, and also, to set 60 000 fires a year to destroy it, and if this fails, to tax the

crop so it cannot be held to maturity, thus ignoring not only the immorality of demanding such contribution to government while confiscating the only chance of reimbursement—crop profit—but also its evident futility. It believes in State and National forestry, but not in appropriations to conduct it. Of nearly 470 000 acres of true forest land in the United States, one-fifth is publicly owned. This belongs to the public to do with as it pleases. One-third is owned by farmers. They have a hard time anyway, so the public will not bother them. This leaves 47% owned by lumbermen, who are assumed to be more unreasonable than the others. Something ought to be done about this! There really ought to be a law compelling them to grow all the trees needed! If they owned the mines, too, they might be made to produce enough gold to make all rich. This is not a fanciful statement of composite public viewpoint, but the writer is unable to picture any economic aspect it surveys with consistency.

The economic aspect of reforestation to those who have the job to do is largely determined by the prospect of reconciling all these views. In most cases starting the crop is simple. It is doing very well starting itself. On the Pacific Coast, where natural restocking is usually swift and certain if given any chance, this is the least of the problems, certainly not the one that deters lumbermen. A large proportion of their cut-over land bears varying aged second-growth timber and little that shows no recovery needs much artificial help except better fire prevention. The problem is mainly of carrying the land, tax, and protection investment for the long term involved, with the risk of destruction before harvest. Bear in mind that private reforestation requires not only reasonable assurance that these obstacles can be overcome, but also that there will be the continuing revenue with which to do it. Since the promise to unrelated capital seeking investment is small, this means a going, profitable lumber operation for the period. In New England, successive crops since original exploitation meet this need for many companies, especially paper companies. On the Pacific Coast the reserves of uncut timber may meet the needs for many companies, but by no means for all. The problem is regionally hardest in the Central States, with little virgin timber or re-established forest. In any region, however, the owner without supply to bridge the gap until he can cut again, can seldom see his way through.

It further becomes an individual problem through the different financing methods of companies; the demands of their stockholders for early or late liquidation or for dividends re-investable in better paying lines, their plant construction, or other ability to use or sell the different products of the new crop; their transportation problems; the size and location of their holdings; the policy of their neighbors; and other things. Again, there is the fundamental question of land quality for growing new crops, or its accidental condition through fire or past error; with other factors bearing on competition with other owners or other regions, or perhaps with Government or State forest competing without interest, taxes, or even the necessity of meeting costs.

Lumbermen Are Attacking Problems

Very conspicuously, long-life operators in the two great producing regions of the country—the South and the Pacific Coast—are passing out of the indifferent drifting stage that always belongs with early exploitation and are setting themselves to the solving of these problems. That it is a joint private and public responsibility has also been expressed as National policy by the Clarke-McNary Law for the building up of Federal, State, and private co-operation to deal with fire first, so other steps may be warranted, and then with taxation, at the same time creating incident spirit and machinery for solving other problems in the same joint way. This law also recognizes, as National forest policy, that the most urgent need is not to govern the cutting of forests still uncut, affecting a comparatively small area in the near future and certain to improve in method, but to foster and protect reforestation on the vast already cut areas, on which the public will first depend, which object is exactly the one that is becoming of keen business interest to the accumulator of cut-over land.

On the Pacific Coast there are more than 9 000 000 acres of privately owned cut-over and burned forest land, the majority in some stage of natural re-stocking. Its area is increasing rapidly. A large proportion is capable, with small effort except protection, of producing another crop in 40 to 60 years. Some areas are well along with this crop. The question with the owner is not whether he shall engage in reforestation. He is engaged in it. He has the land and cannot dispose of it. In all Pacific Coast States except Montana there are compulsory patrol laws requiring him to protect it. Fire prevention has reached its highest development here. The organizations the writer represents spend from $1 000 000 to $2 000 000 per year on it. Therefore, the owner has the land, a crop started, or fairly assured in most cases, a heavy fire bill, and a heavy increasing tax burden. Without much forethought he has been caught in the current and is swimming. He is now calculating how long he can last. Although this is a fair way of describing an industrial situation, no two individuals have the same problem. They vary tremendously as to their own strength, the strength of the current against them, and the distance from shore. Reforestation is no more an abstract calculation than farming, fishing, or horse-racing.

About thirty large companies on the Pacific Coast have gone into the question, perhaps ten of them being committed to continuous production on their lands, and the remainder still investigating. The Research Department of the Western Forestry and Conservation Association has eighteen such clients to date, including some of the largest owners. Great variation is found in economic possibilities, even on similar cost and return estimates, due to differing productivity and condition of lands. Roughly, it sums up that well-along second-growth, if it survives, seems able to stand quite heavy charges because the carry is short; but that much land now being cut, or recently burned, so that it must be carried fifty years or more, will not pay fire and tax charges as now seen. In either case, there is the risk of still

higher taxation and of fire destruction, and some companies have not the operating life to bridge the gap until another harvest.

On the other hand, there is a different way of looking at it; not the traditional forester's way, nor yet the traditional lumberman's way, but a straight business way. Modern business tries to reduce loss and waste to the minimum. Although it is often difficult to show probable profit in private reforestation as an enterprise, it is equally often easy to show certain loss without reforestation as a part of operation. In other words, if there is to be a loss in this land, it pays to reduce it. If as a by-product it has no value to be redeemed for the stockholders, except its forest-producing value, it cannot possibly be left around outdoors, a charge and a fire hazard; without any policy of management, use, or disposal; without gathering needless costs and bringing needless losses. If charges against it are inevitable, they should not be increased but the redeeming earnings should.

Responsibility Must Be Shared

This is exactly where the lumberman is arriving, by combining technical forestry appraisal of possibilities with his own technical operating administration, to make reforestation a business equation. No one knows the ultimate distribution of forest-land ownership. The industry will hold more than it now thinks, but history shows it rarely keeps one-half in any country, the less profitable majority being feasibly carried only by the public. Therefore, it will be here, the ownership and responsibility shifting for many years, under economic pressures, toward a distribution based on what the industry can hold profitably with the remainder going to State and Government either in good condition through purchase or in bad condition through enforced neglect and abandonment. The public burden of the poorest land will increase with the difficulties it imposes on private enterprise and *vice versa*.

However, whether retention or disposal is the ultimate course of any lumberman with all or part of his land, it is obviously his gain to sustain values realizable either by use or by disposal, in order to repay costs meanwhile, to the extent this can be done without too great further expense. This is becoming the policy of the progressive lumberman of the Pacific Coast. He does not know yet to what extent it may be frustrated by the public through fire, taxation, and other lack of reciprocity. He does not know whether the harvest will pay; or whether he, another, or the public will reap it. Meanwhile, however, if possible, it must be better to preserve than to destroy the productivity that is the only, if an uncertain, asset. Toward this end he is studying harder, working harder, spending more money, than any of the other eventual beneficiaries. He is succeeding very well as it is. A little more reciprocity and it will be a go.

REFORESTATION

J. B. Woods,* Esq.

During the past ten years many American lumber manufacturers have investigated the possibility of practising forestry and year after year increasing numbers of progressive operators are taking up these various activities, termed reforestation.

All are agreed that the future welfare of the country demands the working out of some general forest policy whereby private owners and the public may find profit in the perpetuation of productive forests on forest lands. This problem is one of land use—the production of timber trees on lands best suited to such growth.

In the final working out of permanent forestry it may be expected that timber will be produced near the various centers of consumption so that long freight hauls may be reduced to a few miles instead of thousands of miles. It is expected also that the present large areas of stump land will be restored to the beauty and productivity of the early days before the saw and axe.

Leaving aside the esthetic considerations—and they are worthy of serious thought—the commercial and industrial future of this country demands a rapid development of forestry practice. It is also fair to state that social solidity rests on a foundation of accessible and relatively cheap building materials. In other words, plentiful forest materials are essential to the small structure which shelters the typical American family or business. To maintain the supply of cheap but good building materials the forests must be perpetuated by wise use.

Problems and Hazards of Forestry

There are problems, however, before the lumberman who would practise forestry. Taxation is a serious and well-nigh universal problem. The revenue requirements of States and counties are increasing with correspondingly heavier levies year by year on cut-over lands as well as timber and physical properties. Heavy property taxes on standing timber have been instrumental in forcing a rapid depletion of the forests. Similarly, in some sections, excessive taxation of denuded lands has caused many owners to allow them to revert to the State. If forestry is to be undertaken by private capital, taxes must be kept at reasonable levels, and private owners must be able to budget this yearly item of capital expenditure over long periods. To-day, in most States, the tax laws do not recognize that timber is a crop; land taxes are excessive, and there is no way of forecasting the future tax outlay over a period of years to come. Whether timber is in the South or in the Pacific Northwest its worst enemy is fire, and the great American public is chiefly responsible for the increasing yearly toll of loss by forest fires. In the Southern States fire which runs over the ground does not kill the large pines,

* Forester, Long-Bell Lumber Co., Longview, Wash.

as a rule, but fire does kill the millions of seedlings which Nature plants to replenish the forest, and repeated fires impoverish the soil so that scarcely anything will grow. In the Western pine country the effect of fire is similar to that in the South, whereas in the regions where Douglas fir predominates there is danger of crown fires which may destroy virgin timber. Fortunately, the hazard in the Northwest is restricted to the summer months.

Throughout this country the public is foolishly complacent regarding forest fires. Year by year the State and Federal forestry agencies, groups of timber owners, and individuals spend money in protective work and strive to awaken the public to a fire-consciousness, but progress is disappointing and the fire damage increases, and the land owner who would grow a new forest must face the necessity of providing fire protection over a long period of years, largely through his own efforts and with his own money.

Investigation discloses other enemies of growing forests. Hogs at large, insects, fungus diseases, windstorms, floods, and droughts are a few of these. Even coon hunters and timber thieves must be reckoned with and guarded against by efficient patrol for second growth stands containing a sprinkling of timber large enough to make into ties and round products.

One of the mainstays of forest management in Europe is selective logging, whereby tree cutting is carried on according to a pre-determined plan and results in the removal of only the largest trees leaving the smaller ones to grow; or it may be applied to improve the stand by taking poorer trees; and, of course, there are combinations of these selections.

For several years American foresters have endeavored to convince lumbermen that they should adopt selective logging methods, at least to the extent of cutting only the larger trees and leaving the smaller ones to grow. There are serious operating objections to selective logging in Southern timber and on the West Coast, and, unfortunately, neither foresters nor lumbermen in America have developed facts showing the relative advantages and disadvantages. Similarly, reliable and specific information is lacking about many other phases of forest management. For example, there are insufficient tree growth data in many localities of the South and West, and little is known of the economics of thinning young stands of sapling poles.

Virgin Timber Is Merchandising Standard

One very important fact often has been overlooked by those who urge the lumber industry to practice forestry. In the fiercely competitive business of manufacturing forest products, high quality timber is essential. In the past the buying public has demanded the best lumber obtainable, even if cheaper material might have been more suitable for certain purposes, and so long as high grade timber is available the lumberman must meet competition based on the manufacture of such timber. There are purposes for which second-growth timber and even thinnings from young stands are suitable. These uses will increase, but here, again, competition enters—the damaging competition of careless and often irresponsible marginal operators who are in business to-day and gone to-morrow. The established lumber manufacturer

may place some material of this quality which he may develop, in markets where its use will be most satisfactory, but as a rule he must hold and protect this second growth until it reaches commercial maturity.

Meanwhile, in order to maintain the standards of his forest products, the lumberman must secure reserves of high-grade virgin timber to last as long as the supplies of his competitors. He can begin now to prepare for a future wherein man-grown trees will replace virgin forests by applying forestry principles to certain phases of his operation, but he knows that virgin timber will dominate the lumber markets of America for many years to come. It is evident that the application of forestry science to the business of manufacturing lumber is complicated by many economic considerations.

However, progress is being made and certain activities now being carried on by the writer's company will indicate what can be done. Taxation is a serious and almost universal problem. On the Pacific Coast some progress has been made in formulating legislation to encourage timber growing by lightening taxes. A proposed amendment to the Constitution of the State of Washington, if ratified by public vote, will form a basis on which to frame constructive tax legislation for the future. Meanwhile, a program of reforestation has been formulated at Longview and Ryderwood, based on the proposition that the voters will give constructive tax legislation which will permit the growing of trees over a long period of years.

Longview Reforestation Plan

The underlying idea of this plan is to re-seed the cut-over land at a rate equal to the progress of denudation. This probably will range from 2 000 to 3 000 acres per year. If one could be sure that Nature would re-seed the cut-over lands, it would be very simple for the forester. He would merely protect the lands from fire and wait for the trees to start, but natural restocking is a tedious process often accupying several years. As interest on land investments, taxes, and other carrying charges must be taken into account, it is evident that the forester must take steps to get his new forests started if he expects the trees to reach merchantable age and return any income to the owner. Compound interest is his enemy.

The idea at Ryderwood is to wait two years for Nature to begin her own process of re-seeding, then it will be possible to tell where natural seeding can be expected to develop favorably. Where Nature does not appear to be able to start anything, nursery grown stock will be planted. An area cut over in the winter of 1926 would be planted in the early spring of 1928 after having been slash-burned either in the spring or fall. A word should be said about slash-burning. There is a statute of the State of Washington requiring owners or operators of logging enterprises to abate the nuisances left by them in the form of logging débris. The law does not stipulate that this shall be burned, but there is no other way that it can be removed and abated to-day at reasonable cost; therefore, it is necessary to burn all cut-over lands after logging and before reforestation can proceed. The question of advisability of such burning may be open to argument but slash-burning is carried on

regularly and with all possible precautions to prevent undue damage, outside the slash-covered area.

It is sincerely hoped that the next few years will bring about developments in wood utilization which will make possible the abatement of slashing nuisances through pulping or wood distillation and without the use of fire on the land.

Forest Nursery Maintained.—Although it is hoped that natural seeding will account for two-thirds of the cut-over area, nursery facilities are provided for growing enough plants to re-stock two-thirds each year. The nursery maintained at Ryderwood has a capacity for the continuous production of 2 000 000 seedlings per year. A tentative growth period for this new forest has been set at 50 years. On soil such as this forest will occupy, it is expected the growth in 50 years will amount to 20 000 B. M. per acre on the poorest sites and probably 40 000 B. M. per acre on the best. Probably after 25 years it will be desirable to enter these stands and remove enough poles and small trees to thin and improve the growing conditions. One of the many places that engineers can help conserve timber and encourage reforestation is in the adoption of creosoted or treated-wood products specifications of "construction materials". In the South where the writer's company maintains several creosoting plants it is possible to thin growing stands of pine at profit because this raw material is made into fence posts, telephone and telegraph poles, piling, and ties and the Company is willing to pay a satisfactory stumpage rate for it. By treating such materials the period of useful life is extended five to tenfold. Twenty years ago such a development was rarely found in the Southern States. Therefore, it is likely that the next twenty years will witness a growth of the wood-treatment business in the West and that when stands of growing Douglas fir are thinned a market will be found for these products in the construction field.

Improvement of Species.—While working out the program of starting new forests for Longview, it is felt that every reasonable effort should be made to improve the quality and variety of materials for the future cut; therefore, experiments with redwood, white pine, and Port Orford cedar are being made to supplement Douglas fir. Douglas fir is the most valuable wood for all-around use, but there are sites on which other species will do better than the fir and on these it is expected to grow redwood and Port Orford cedar and possibly some white pine. If this plan can be carried out, a balanced future forest will be obtained with Douglas fir as the principal species and certain other well-known trees for special purpose woods.

In addition to these species, it is expected that the new forest will contain large quantities of hardwood, chiefly alder, which attains large size on the Coast. Alder seedlings are planted along abandoned rail spurs to form fire screens for the protection of the stands of young conifers. The plantations are thus subdivided into compartments and the fire hazard is correspondingly diminished.

Naturally, fire protection is the prime requisite of reforestation. In addition to timberland patrol with its 100 miles of trail and telephone line, its

lookout and patrol stations scattered throughout the woods, there is the camp patrol maintained by the Logging Department, and a special patrol for the new plantations of Douglas fir, redwood, and Port Orford cedar. All this is at the Ryderwood operation. In other parts of the country the writer's company maintains eleven more fire-protective organizations watching virgin timber reserves, logging operations, and young forests.

Other Forestry Activities

Forestry activities are not confined to the State of Washington. In Southern Oregon about $200 000 has been expended by a group of private owners in co-operation with Governmental authorities to arrest and control the ravages of certain pine tree killing beetles. Despite disappointments as a result of repeated years of adverse climatic conditions which have favored the beetles by retarding tree growth, progress is being made.

In Arkansas, Louisiana, and Texas, the Company has set aside more than 200 000 acres of forest reserves where young timber is growing under the best care and fire protection that can be given. In Western Louisiana another forest nursery has been started to provide planting stock for a large program of re-stocking cut-over lands with longleaf and slash pine. It is hoped that in time the Company shall have there a naval stores enterprise similar to the hand-grown forests of Southern France, and, meanwhile the production of posts, poles, and ties will be continued for the treating plants.

Experiments in selective logging are under way in California. This involves replacement of steam skidding equipment with caterpillar tractors and two-wheeled carts, in order to save the young timber and the seedlings, but to be worth while such experiments must be made on a large scale and over a considerable period.

In common with other large operators the Long-Bell Company is attacking these reforestation problems aggressively, but with caution. Lumbermen must have help to work out such solutions as will benefit the public as well as themselves; they must have constructive tax legislation in all timber-producing regions and the helpful fire consciousness of an awakened public; and they must have a better realization of the merits of wood as a material for construction and for fabrication into other products, and through this channel a larger measure of wood utilization and less waste. The manufacturer of forest products who cannot do business profitably and who must leave much of his raw materials to rot on the ground for lack of a market will be too deeply harassed by present difficulties to devote much time to thinking about growing trees for future generations.

SOME PHASES OF IRRIGATION FINANCE*

By D. C. Henny,† M. Am. Soc. C. E.

The arid portion of the United States is to a very great extent dependent for its development on irrigation. It must grow the bulk of its own food requirements in order to permit industries to grow and its resources to be developed. In the earlier years of far Western settlement, mining was the principal industry and afforded sufficient excess profit to stand the burden of high food prices resulting from expensive transportation. With the exhaustion of exceptionally rich mining fields and the advent of other industries, growth has progressively become more dependent on local food supplies.

The part of the United States most directly affected by irrigation is comprised of the States of Wyoming, Colorado, and New Mexico, and those lying farther West. The importance of this region to the nation is due not solely to its own development, but especially to the far-reaching influence which a populous West will have on the future of the United States and the degree of pressure it will be able to exert in the solution of international problems surrounding the Pacific Ocean.

The population of the West has been growing faster than that of the country as a whole. The area under irrigation has shown even a relatively greater growth prior to 1910 than the population, as is indicated by Table 1, derived from United States Census reports.

TABLE 1.*

Year.	Population.		Irrigated Area, in Acres.	
	Total.	Percentage of United States.	Total.	Per capita.
1890	3 102 000	4.8	3 555 000	1.14
1900	4 101 000	5.4	7 253 000	1.76
1910	6 826 000	7.4	13 202 000	1.94
1920	8 903 000	8.4	17 401 000	1.94

* In this, as in the following tables, large figures are expressed to the nearest thousand.

The number of irrigated acres per inhabitant was small during the early mining years. By 1890 it had increased to 1.14 and since that time it has continued to grow, so that in 1920 it had become 1.94. From 1910 to 1920 there was no increase in this relation, which appears for the time being to have become stabilized at about 2 acres per inhabitant.

* Presented at the meeting of the Irrigation Division, Seattle, Wash., July 15, 1926.

† Cons. Engr., Portland, Ore.

The tier of States reaching from North Dakota to Texas was excluded intentionally from these statistics. Along the western and southern borders of this area irrigation is of great importance. The eastern border, however, penetrates the humid zone and figures for all the States include preponderating population and farming areas not dependent on irrigation.

It is well known that a considerable portion of irrigation enterprises remains uncultivated for a long time after they are opened to settlement. Mr. R. P. Teele has presented* the figures in Table 2, which are based on U. S. Census reports, relative to this statement.

TABLE 2.—Extent to Which Estimated Full Capacity of Irrigation Enterprises Is Utilized at Various Periods After Commencement of Construction.

Years.	Percentage.	Years.	Percentage.
5	36	25	60
10	45	30	62
15	52	35	63
20	56	40	65

At first sight the condition revealed by Table 2 will be regarded as exceedingly unsatisfactory and as showing serious irrigation over-development. It should be noted, however, that the term, "Estimated Capacity of Irrigation Enterprises", has a rather uncertain meaning. Mr. Teele points out that it represents the hopes of owners rather than actual irrigation possibilities.

If tardy settlement and over-development are under consideration, the area irrigated may best be compared with the area which could have been served with an irrigation supply. This has been done in Table 3, the figures covering all States west of a line drawn from Galveston, Tex., to Grand Forks, N. Dak., thus including all States grouped by the U. S. Census Bureau in its irrigation statistics, except Arkansas and Louisiana.

TABLE 3.

Year.	Area enterprises were capable of irrigating, in acres.	Area irrigated, in acres.	Percentage of area irrigated.
1890	Not reported.	3 631 000	......
1900	" "	7 519 000	......
1910	19 685 000	14 025 000	71
1920	25 112 000	18 593 000	74

Table 3 tends to show that in 1920 about 6 800 000 acres were lying idle to which existing works were capable of supplying water, an area more than one-third of that actually irrigated. Although this showing is not as bad as that implied by Table 2, it would indicate nevertheless a serious condition of

* *Bulletin No. 1257*, U. S. Dept. of Agriculture, 1924.

over-development. It is believed that these figures, however, are also strongly colored by optimism. The reports for Oregon, for instance, give an area of 350 000 acres to which water could have been served but which was not irrigated. The writer's familiarity with the State inclines him to the belief that this figure is grossly in error.

The reports of the United States Bureau of Reclamation are undoubtedly more accurate than the general average. These reports give for 1920 a corresponding 75% irrigation. The average age of settlement on Government projects is far less than that on the entire irrigated area of the West and for the same age a considerable higher percentage is likely to be found as compared with 74% for the whole, as given in Table 3. There may be, however, some difference in favor of Government projects as offering superior inducements.

Before drawing conclusions in a matter of this importance it may be well to consider that the partial settlement of irrigation enterprises, were it correctly known, is not in its entirety a measure of excess development. The first settlers generally select the best soil and the remnant available to new settlers leaves lesser possibilities of profitable farming. The water supply may have proved unsatisfactory for the full area reported as irrigable, some land may have, or threatens to, become water-logged and even in the most intensively farmed areas some land is always found unused due to change of ownership, failure, and other causes.

If from these considerations, 85 to 90% may be judged as the normal maximum it may be reasonably estimated that from 1 500 000 to 2 000 000 acres to which water could have been furnished remained uncultivated in 1920. To what extent this area may consist of land which because of roughness of topography or character of soil remains unused is very doubtful. Some of this land cannot place a new settler on a competitive parity with other settlers on land in the same locality until a sufficient price difference has developed and as this, in turn, depends on crop prices, margin of profit, and availability of better land elsewhere, a large part of this land may have to be considered as not available at the present time.

A considerable slack has probably always existed and cannot in itself be regarded as evidence of over-development. In any expanding industry there is usually provided a margin ahead of pressing demands. The question of over-development of irrigation has only been generally raised during periods of agricultural depression, which affect the East and West alike. The severity of the present agricultural depression may be judged from the changes in the area of improved farm land in successive decades, as reported by the U. S. Census Bureau (Table 4).

It is of the utmost importance in connection with all irrigation finance to understand as fully as may be the causes which have contributed to this depression in order to have some judgment of its probable duration. This is especially true when dealing with reclamation projects on such scale as is now becoming unavoidable, requiring many years in the process of mere building and possibly decades in the course of their normal development.

Low crop prices are the direct cause of agricultural depression and are due mostly to over-production resulting either from lack of purchasing power on the part of the consuming public or to over-development. It cannot be said that there is any lack of purchasing power at present in the domestic market; on the contrary this is probably at present higher than it has ever been, all industries other than agriculture being in a reasonably prosperous state. On the other hand, the foreign market is in an unsatisfactory condition as regards absorbing any normal surplus which is usually exported. This is well illustrated by the fact that of the 31 000 000 acres decrease in improved farm land area from 1920 to 1925, wheat land accounts for 22 000 000 acres, this being about 30% of the total area devoted to wheat raising in 1920. During the same period, however, the area in cotton, perhaps the next most important article of export, has increased more than 5 000 000 acres, or about 15%, which in part accounts for the relative prosperity of far Southern agriculture until the recent slump in the price of cotton.

TABLE 4.

Year.	Land in improved farms, in acres.	Increase, in acres.
1890	623 000 000	....
1900	839 000 000	116 000 000
1910	879 000 000	40 000 000
1920	956 000 000	77 000 000
1925	925 000 000	-31 000 000

On the whole, it is probable that over-production in relation to available markets is the outstanding cause of the present depression. As far as over-production is local, it may in part be mended for those products which can stand long-distance transportation cost by reaching out for more distant domestic markets, as has been successfully done in the case of citrus fruits and, later, of deciduous fruits.

Farm production is as yet entirely uncontrolled and unrestricted. Good prices at once cause increased production, the tendency being always toward over-production in the industrial, as well as the agricultural, field. In the former, production is in fewer hands and intelligent forecasts have their beneficial effect on output control. If large steel corporations should rapidly increase the output of certain steel products, simply because the margin of profit was high during the previous year and without reference to the absorbing capacity of the market, they would soon be in no better financial condition than the farmers.

Restriction of output to correspond with probable demand is as necessary to maintain the agricultural industry in a satisfactory condition as it is for other lines of industry, but is far more difficult of realization. Uncontrolled output results in violent fluctuation of prices with the ever-recurring succession of periods of inflation and deflation. These, in turn, produce a similar effect on land values with the aggravating subsequent combination of heavy

investment and low crop returns. The writer visited Europe a few years ago and found that, in Holland, shrinkage in exports had resulted in over-production, causing a serious agricultural crisis. On the other hand, farmers in France and Germany were relatively prosperous, the effects of the World War having caused under-production.

If economic laws are not interfered with by legislation, an agricultural setback has at least this beneficial feature—that it causes marginal lands to go rapidly out of cultivation. In the meantime the growth of population maintains a rising curve in the domestic demand and it is merely a question of time when it will cross the falling curve of output. When this happens crop prices will rise, the farmer's prosperity will return, and the demand for farming land will revive. Artificial revival can be brought about by legislation. Legislative interference with economic laws is believed to be justified only under exceptional conditions and for distinctly temporary purposes. Price maintenance is extremely costly to the nation as a whole, because it aggravates the original cause by encouraging production at a time when the demand for price maintenance is created by over-production.

The importance of stabilizing crop prices when once they have reached a point yielding profits in proportion to other industries cannot be exaggerated provided it can be done without violating economic laws. It is possible that some healthy progress may be made in that direction by the distribution of reliable statistical information and by the forming of farmers' organizations leading to output restriction, in all of which Governmental action may be helpful. Upon recurrence of profitable crop prices there will develop, as far as irrigated agriculture is concerned, an immediate tendency to take up the slack by occupation of the best farming sections of the 1 500 000 or 2 000 000 acres for which water is reported to be ready. This can be done without starting new irrigation projects. If about one-half this area is destined to come into early use and if the eleven Western States should continue to increase in population as they did in the last decade at the rate of 240 000 per year, requiring an increase in irrigated area of possibly 460 000 acres, it is evident that the present available excess may be absorbed with surprising rapidity.

Most irrigation projects require two years and more for construction, and some of the larger ones need in excess of four years. Ordinary human foresight, therefore, would indicate that the desirability of energetically resuming irrigation construction may become apparent before long. The present time, therefore, is not inopportune for inquiring into the various means by which expansion is most likely to be brought about and into the manner in which it will probably be financed. The agencies through which irrigation has been accomplished in the past may be judged from Table 5.

Irrigation effected by individuals, partnerships, and corporations, accounts for by far the bulk of accomplishment to the present time. It is probable that through these agencies, in numerous localities where improved transportation removes previous obstacles, small areas will continue to be developed, which, in their aggregate, will continue for a time to be an important factor

of irrigation expansion. Since 1910, however, irrigation fostered by legislative action or carried on directly by Governmental agencies has been gaining rapidly in relative importance. The acreage percentage thus reclaimed more than doubled during the decade 1910-20, while the gain in area was 2 500 000 acres as against 2 250 000 by other agencies.

TABLE 5.

Agency.	1910.		1920.	
	Acres.	Percentage of total.	Acres.	Percentage of total.
Reclamation Bureau.......	396 000	2.7	1 255 000	6.5
Indian Service..............	173 000	1.2	285 000	1.5
Total, Federal............	569 000	3.9	1 540 000	8.0
State........................	Not reported	......	6 000	0.003
Governmental..............	569 000	3.9	1 546 000	8.0
Carey Act..................	289 000	2.0	524 000	2.7
Irrigation districts.........	529 000	3.7	1 823 000	9.6
Sub-total............	1 387 000	9.6	3 893 000	20.3
Other agencies...............	13 046 000	90.4	15 299 000	79.7
Total..........	14 433 000	100.00	19 192 000	100.0

The largest proportionate increase was that produced by the U. S. Reclamation Bureau, its percentage in 1920 being 2.4 times that in 1910. The largest acreage increase due to any single agency was brought about by irrigation districts, amounting to 1 300 000 acres during the decade. The Carey Act, passed in 1894, which at first held out great promise, has thus far accounted for less than 3% of the total irrigated area. It grants to each of the arid States 1 000 000 acres of desert land subject to provision for reclamation by the States. This has been done by State contracts with construction companies which provide the money for and build the works, these companies being authorized to sell water rights to parties buying the land from the States.

The Carey Act contains a sound financial principle in that it permits invested capital to earn, besides interest, a profit which may be commensurate with the risk involved. The unsatisfactory feature in its application was that companies willing to undertake the work were poorly financed and depended on advance water-right sales and the proceeds of bond issues not secured by the land, for a large part of their construction funds. The States of Idaho and Wyoming are the only ones that have largely availed themselves of this method. A few well-selected projects were reasonably successful. Others failed through insufficient water supply or from tardy settlement, causing the financial plan of interest and bond repayment to be upset, with the general

result that this method of irrigation expansion is at present dormant. There is, however, no fundamental reason, except possibly inavailability of suitable public land projects, for discarding its use in the future especially if State legislation can be changed so as to permit the land itself to become security for bond issues. It is safe to say that investors will not avail themselves of this Act until a large margin of profit can be calculated, and this cannot be hoped for until crop returns and land values show a considerable advance over present conditions.

The irrigation district method of developing and handling irrigation deserves the most serious attention. From the standpoint of total acreage and recent increase of acreage, it exceeds in importance all direct financing by Governmental agencies. From the social standpoint, its comparative independence of Governmental aid and control is strongly in its favor. Dependence on commercial financing through the sale of bonds has, in addition, the advantage that selection of locality is not influenced by political considerations and that over-development is to some extent automatically checked. In considering this subject, distinction should be made between the various enterprises which now operate under the district plan. These divide into two general groups: (1) "A" Districts with land values greater, and (2) "B" Districts with land values smaller, than the face value of the proposed bond issue.

It is apparent that where the immediately existing land values, independent of any speculative element, leave a large margin over the amount of the bond issue, and other conditions are favorable, the security is good. This is generally the case with districts which have developed prosperously under some other plan and which are organized and issue bonds for taking over existing and proved works and for improving them; also, with many districts on Government projects. It is equally apparent that where the aggregate land values are far below the bond issue the case is essentially different, the security is uncertain and becomes dependent on the numerous factors which may lead to relative success or failure. There are many risks arising from possible under-estimated cost, insufficient water supply, faulty engineering, and over-estimated rapidity of subdivision and settlement.

The bond-buying public has no means of determining for itself the risks involved and, in purchasing bonds, depends on the standing of bond houses and on their representations. On the whole, bond houses have responded conscientiously to the call thus made on them by a thorough investigation on their part and by refusal to handle bonds involving excessive risks. Risk in greatly varying degree is, however, always present in the absence of an ample margin of security independent of future development. There have been many cases, of course, where bond dealers of the highest standing have misjudged the risk and also where less conservative dealers have handled doubtful issues.

This subject has been ably and exhaustively discussed by Mr. Teele in *Bulletin No. 1257* of the U. S. Department of Agriculture, already referred to, which for unbiased and comprehensive information deserves more publicity and distribution than it appears to have received. In this *Bulletin* a division

between districts is made along somewhat the same lines as that suggested. It distinguishes between Class I Districts organized for taking over projects developed by other agencies and Class II Districts organized for developing new projects. Class II Districts may thus be regarded as largely identical with the writer's Class B Districts, except that the former excludes, and the latter includes, districts which, although partly developed, show an unsatisfactory relation between existing land value and face value of bonds. On the basis of his division, Mr. Teele reports the results given in Table 6, after eliminating all districts which did not pass beyond the preliminary stage.

TABLE 6.

Districts.	Number.	Gone Out of Business.	
		Number.	Percentage.
Class I	246	48	19.5
Class II	156	110	70.5
Total	402	158	39.3

The showing made by Class II Districts is extremely bad and if doubtful security districts in Class I had been included in districts in Class II, so as to make it identical with Class B, it would be still worse. On the other hand, the results for Class I Districts if so amended and made the same as those for Class A, would be very good as the percentage of failures would have dropped probably below ten and possibly might be close to zero. This might be so even if the percentage of failures had included districts which have not gone out of business, but which have defaulted on their obligations.

There are exceptional cases in Class II Districts where the risk in bond investment may be sufficiently paid for by the rate of interest which these bonds carry and by the price at which they are sold, as, for instance, small districts with choice opportunities and with relative cheap irrigation supply. Again, there are instances where the risk in bond investment is deliberately incurred by parties likely to benefit in other ways by land development. On the whole, however, it must be concluded that the district bonding method is not well adapted to the reclamation of unimproved or only slightly improved land. It is mainly for this reason that efforts have been made to throw Governmental safeguards around district bond investment.

These efforts have consisted of public investigation of feasibility, requirement that bonded debt does not exceed 50% of the value of land and works, provision for certification of bonds by State Commissions, and limitation of price at which bonds may be sold. In addition, some States authorize the inclusion in the bond issue of interest for the first few years, and Oregon permits a guaranty of interest for a maximum of five years. Measures of this kind are helpful only if sound judgment is used in their application, which unfortunately has not always been the case. Pronouncing a project feasible although its success is quite uncertain and the security it offers as a

bonding proposition is very unsatisfactory, is an action likely to be taken by a sympathetic State board. To certify the face value of the bonds to be no more than 50% of the value of lands and works when this can be true only, as in the case of desert land, on the basis of speculative or potential land value, is far more questionable. These things have all been done. The Oregon State interest guaranty has been granted with more genuine care but, nevertheless, at times with prospects of loss to the State, and bond buyers have probably attached too great significance to this feature as well as to State certification often prominently displayed on the face of each bond and likely to imply a far-reaching but non-existent State obligation.

All these attempts are for the purpose of helping out a situation which in the case of Class B bonds is unsound, while in the case of Class A bonds they are largely unnecessary for the purpose of financing although they may be beneficial. If reasonable excess security is lacking at the time the bonds are issued, bond buyers run a risk for which the permissible rate of interest and of selling price does not ordinarily offer just compensation, a fact which cannot be changed by law in any way except by flatly placing the credit of the State or Nation behind the district.

The belief that the lien which a district holds on the land, permitting its forced sale when taxes remain unpaid, is of special value to holders of Class B district bonds has often been rudely shaken in practice. When relatively large areas are in default the reason is not a personal but an economic one which cannot be helped by change of ownership. The general obligation clause is also of little avail in such cases because it leads to undue concentration of assessments on lands remaining in cultivation, which is sure to add to the area in default and to result in further decrease of cultivation and of bond security.

In the matter of Class A Districts the district laws have operated with gratifying results. The success attained would be still greater if Class A Districts could be removed from the cloud which the failure of Class B Districts has raised as to district bond investment generally. Class A bonds are in their nature improvement and not development bonds and if this distinction can be brought fully home to the investing public, it should be possible to sell such bonds at a better price or with a lower rate of interest. To some extent this has been successfully done in California and Idaho.

There now remains to be considered irrigation through direct Governmental financing, accounting in 1920 for 8% of the total development. Irrigation has been accomplished to a small extent by the three Pacific Coast States. The experience to date has not been up to expectation. In Washington, one small project was started with funds provided by the State Reclamation Service. Heavy reservoir seepage has compelled a severe cutting down of the project area, and the State stands to suffer a heavy loss. In Oregon, a project started with money appropriated by the Legislature for the purpose has met with a similar although somewhat less disastrous fate, also by reason of reservoir leakage. It has recently been supplied by the State with additional water and there is a probability of reasonably good results being secured,

the State, however, writing off a heavy loss unless the reservoir can be made tight at no excessive expense. In California, two State projects have been built both with State aid in partial financing of settlers. The Durham project may be regarded as wholly successful. As to the Delhi project there appears to be great difference of opinion, it being uncertain to what extent heavy arrears in payments to the State should be fairly attributed to adverse agricultural conditions.

Federal irrigation has been developed in two directions, both controlled by the Interior Department. The Indian Service is doing very important work, which, however, is apart from commercial irrigation as it is intended primarily for the benefit of the Indians and is to be only indirectly judged by financial results. The writer has been unable to ascertain the full facts from which the benefit to the Indians and to the Nation can be judged. Within the limited scope of his own observation it is found that the Indians themselves do not take readily to intensive irrigation farming and in some reservations a large part of the land under ditches lies uncultivated. Where the lands are attractive to white settlers and can be leased by them, the results are comparable with those obtained by tenants on U. S. Reclamation Bureau projects, the Indians merely benefiting as landlords.

The work of the Bureau of Reclamation has been repeatedly the subject of public investigation. Reports have been made by committees of Congress, by an Army board, and recently by the so-called Fact Finding Commission. These reports enter into great detail and have received considerable publicity for which reason the writer proposes to deal only with a few general features.

In judging the working of a law like the Reclamation Act, applied under a remarkable variety of climate, crops, transportation facilities, and cost of farm help, certain broad facts should be borne in mind, some of which affect agriculture in general, whereas others pertain solely to Western agriculture and, again, others are of a political and psychological nature.

Much has been heard of the inability of farmers on Government projects to earn a living, of the hardships and misery suffered by them, and of projects being abandoned. It may be presumed that through all the published reports and through direct observation the Western public and its Congressional representatives are fairly well informed as to the general results achieved. In the face of the severe criticism which has been leveled at the Reclamation Bureau, it is a most significant fact that from all the Western States there is a persistent clamor for more Government projects and that this clamor is ably and vigorously supported by their Congressmen. The demand, moreover, is for new projects on an increasing scale of magnitude and is so general and unanimous that it cannot logically be ascribed to motives reminiscent of "pork barrel" appropriations. If sufficiently informed the public is not readily carried away by partisan commendation or condemnation. As regards the alleged severe suffering of Government project settlers, the desire for more from each locality where the facts should be most fully known, carries rather complete refutation.

The reports made so far are unanimous in approving and commending the technical side of Federal irrigation. Criticism has been made of the cost of the work having been under-estimated. There have been of course many cases of erroneous judgment, much of the work being of a novel nature and in partly inaccessible localities. The excess cost complained of, however, has been largely due to a persistent and unforeseeable rise in prices of labor and material and to the addition of numerous construction features not originally contemplated. To that extent criticism on this score is believed to have been unjust.

It is undeniable, however, that the financial results of the Reclamation Act from the direct Government investment angle are not what had been originally expected. The Act as first passed required the repayment of the estimated construction cost without interest in ten equal annual payments. Payment of 10% of construction cost per year, in addition to operation and maintenance charges to be paid by settlers on raw desert land, is indeed a task which, as is now fully understood, only a few could perform.

British Indian irrigation is often and with justice pointed to as an example of efficiency and financial success. There are enormous differences between British India and the United States as to economic and climatic conditions, density and character of population, and need of additional land. A direct comparison is, therefore, not practicable. Nevertheless, it is instructive to note where the Public Works Department of British India draws the line between financial success and failure. The criterion is whether within ten years of the completion of construction a project produces sufficient revenue to cover its working expenses and 5% on its capital cost. Although the aggregate results have been very satisfactory there were, out of 64 productive projects, 15 which failed to meet this test.

Irrigation district, Carey Act, and corporate experience in the United States is conclusive to the effect that the repayment requirements of the original Reclamation Act were too severe and that as a test of success or failure the inability of most farmers on Government projects to make such payments has no value. The inevitable result of these requirements was a search for the easiest way out. This was found in the discretionary power of the Secretary of the Interior to defer the time when payments should commence. That this means of escape was badly stressed even after other remedies were provided is evident from the fact that on some of the most successful projects more than ten years elapsed after water was first turned on before construction repayments began. The next result was that by successive changes in the law, annual payments were reduced to 5% or, under certain conditions, to a graduated scale beginning with 2 per cent. Lately, this has been further changed to 5% of the gross crop returns.

The comparative ease with which these and other relief measures were secured from Congress, the fact that not in a single case, even after the passage of the newer Acts, has the Government enforced collection by foreclosure under lien, and the ready encouragement of farmers' demands by Congressmen, have had an unfortunate effect on the psychological condition

on some of the Government projects and on financial returns to the Reclamation Fund. It is to the great credit of the officers of some of the Water Users' Associations and Districts that payments have been made without demur and with a small percentage of arrears. Under these conditions it is also deemed rather creditable that out of \$28 000 000 due under the new Act for construction charges nearly \$23 000 000 has been paid. About the same proportion of maintenance and operation charges, namely, \$14 500 000 out of \$18 000 000, were collected, all as of June 30, 1925.

The economic features of Reclamation projects have lately been called to the attention of the public with entire propriety. This appears, however, to have created to some extent at least the erroneous impression that the essential elements leading to success or failure were neglected or entirely disregarded in previous planning of irrigation development. To that extent injustice has been done to earlier workers in this irrigation field. It was as well known twenty years ago as it is now that irrigation cannot be successful unless it brings suitable financial rewards and that such rewards are possible only where reasonable crop prices combine with good soil, a sufficient water supply, proper drainage, and individual industry, experience, business ability, and adequate financial resources. It was realized that requirements are high, but that they are in most essentials not different from those required in agriculture generally and in other gainful pursuits. An average percentage of failures therefore could be reasonably expected on Government projects. The number of failures has never been determined and it is, therefore, an open question whether their number is greater or smaller than in other occupations, in other irrigation enterprises, and in other gainful pursuits. It is by such determination only that results can be fairly appraised.

Since the beginning of irrigation, suitable soil and sufficient water supply have been recognized as essential factors. Soil examination by agricultural experts, as far as the writer is aware, has always preceded approval of Government projects. The knowledge of soils is advancing rapidly and much has been learned from past mistakes so that the chance of erroneous judgment is constantly lessening. As to the desired qualifications of settlers, it has been regarded as good policy for many years, especially by private irrigation managers, to select the proper kind of men. It has been found exceedingly difficult, however, to carry this policy into full effect. It has indeed been a rare occasion when any irrigation settlement scheme was confronted with a plethora of settlers from whom to select, even if an abundance of speculative investors has at times been in evidence. On the contrary the case has generally been quite the reverse, land waiting for settlers and irrigation companies making expensive efforts to sell and colonize. Under such conditions the turning away of prospective settlers no matter how poorly equipped has been frowned upon both by the community itself and by the interests in control.

On projects fostered by State or National Government the financial stress is not so immediate, but in many private irrigation enterprises the need of money has been often so great that immediate income from any settler who

offers to buy, must be accepted. The exaction on Government projects of certain qualifications which a settler has to meet is based on sound economics, but in this case, too, a policy of careful selection may be found extremely difficult to enforce. A common argument for securing public funds for irrigation has been that it furnishes homes for the poor. It was only a few years ago that the late Secretary Bellinger was severely and widely criticized for stating at an Irrigation Congress that a poor man had small chance to succeed on a Government project.

If an applicant for land is in earnest but is turned away, the person responsible therefor will probably have to make repeated explanations to Congressman, Senator, or Land Board of his ruling against a worthy Republican or Democrat, as the case may be. The result, of course, should be a strict adherence to the approved policy but, unfortunately, it is likely to be an easing off or even a practical suspension of restrictions unless public opinion can be fully awakened in its support.

Lack of settlement control produces undesirable results. Yet the economic law of the survival of the fittest tends to provide a slow cure. The settler who cannot succeed for reasons inherent in himself will fail, and the sooner he engages in some other pursuit and releases his land the better for all. If a settler of that type is coddled by easy Government credit or continued release from obligations the economic law referred to, ceases to operate and both physical and psychological conditions of the community are adversely affected. To this cause is due in large part the lack of complete success of Federal irrigation. It does not result from the attitude of any man in control but from conditions beyond any one's control where business and politics cannot be kept strictly apart. This is the most valid argument which has been advanced against Federal irrigation and might be conclusive if a continued healthy expansion could be brought about by private enterprise, unless some remedy can be found which will keep politics out of Government irrigation.

This may not be altogether an irridescent dream. There may be ways in which progress is possible. For instance, a very valuable suggestion was made in that direction by Mr. R. E. Shepherd,* who advanced the proposition that Federal irrigation be placed in the hands of a corporation created by the Government, connected with, but not controlled by, the Departments of the Interior and Agriculture. It is unbelievable that when the absolute need is apparent careful thought and discussion should fail to find some satisfactory solution.

The Reclamation Act has been frequently alleged as constituting a subsidy. The Teele *Bulletin*, previously referred to, states: "There is no justification for a National subsidy to land reclamation. If local interests justify the subsidizing of land reclamation the subsidy should be local." The writer cannot fully concur in this conclusion because he believes it is too sweeping. The great National interest in rapid development of the West has induced railroad subsidies on an enormous scale. Cities on deep water have received subsidies in the shape of funds for harbor development. Import tariffs are

* *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 403.

in the nature of subsidy to certain industries. The extensive and valuable work of the U. S. Department of Agriculture in establishing experimental farms, giving free agriculture and even engineering advice, and distributing needs, is essentially a subsidy to agriculture. It cannot be denied that some, if not most, of this subsidial assistance has been of great benefit to the Nation. Subsidy as such is not necessarily unjust or harmful although it readily becomes so if extended unwisely or too freely.

Material expansion of irrigation involves ever-increasing expenditure per acre, as it can depend only on residual stream flow necessitating relatively greater outlay for storage and because it usually requires longer average distance of water carriage. In a broad way the rising cost is confirmed by Table 7 based on U. S. Census reports.

TABLE 7.

Year.	Cost per Acre.	
	Construction.	Annual maintenance and operation.
1890	$ 7.96	......
1900	9.04	......
1910	15.85	$1.07
1920	26.81	2.43

As is well known, the average construction cost as reported by the U. S. Census Bureau gives no fair conception of the cost of projects now being contemplated and executed, which vary from $75 to $175 and more per acre. The averages reported are low because they include the construction of a great number of early simple systems free from the necessity of storage, that were often built in the cheapest and most temporary manner and with the land close to the source of supply.

Table 7 might be revised to bring out somewhat more clearly the growing cost of irrigation. This is done in Table 8.

TABLE 8.

Year.	Area covered by projects, in acres.	Total cost.	Increases of area, in acres.	Increase of total cost.	Cost per acre of increased area.
1890	3 710 000	$ 29 534 000	3 710 000	$ 29 534 000	$ 8.15
1900	7 744 000	70 010 000	4 034 000	40 476 000	10.40
1910	20 285 000	321 454 000	12 541 000	251 444 000	20.10
1920	26 020 000	697 657 000	5 735 000	376 203 000	65.60

No accuracy can be claimed for the figures given in Table 8. The acreage is that which enterprises reported as capable of irrigation and, for reasons previously stated, this may have been seriously over-estimated, while for the practical irrigation of some of it, heavy additional outlay may be required. The figures, however, are of undoubted value as showing the rising trend.

The limitations of desert land reclamation by private finance under such conditions have been discussed. The question of helping out by local subsidy was raised by Mr. Teele who presumably refers to State subsidy. So far, the work done directly by States is very small, but it is possible that this method of financing irrigation may be utilized in the future to a greater extent, especially if in this case also the baneful effect of mixing business and politics can be avoided.

It should be remembered that most Western States are highly taxed not only because the standard of life is high, but also because a large portion of the land, in some States well exceeding 50%, continues in public ownership and pays no taxes. Much of this land is in forest reserve and in Indian reservations and has great value. It should be apparent, therefore, that although some States have made courageous efforts, no great results can be hoped for in that direction, and it is evident that Western irrigation and, consequently, population will advance slowly unless Federal aid on an increasing scale can be extended. If the belief should prevail that continued rapid growth of the West is for the good of the Nation, whether for international reasons, or because it contributes heavily to Eastern industrial prosperity and to the well-being of the transcontinental railroad system, or because it adds to the value of Government land, or for any other sound reason, a certain help from the National Government even if termed subsidy may be fully justified.

If it be conceded that National aid to Western irrigation is wise and should be extended in proportion to the need as it may develop, then it is important that any commitment which the Government may undertake, be known in advance as far as practicable, so that it may be fairly compared with the expected benefit. All tendency to minimize deliberately or carelessly such commitment should be definitely discountenanced.

In the past, construction cost estimates of U. S. Reclamation Bureau work have been as fair as engineering ability and integrity could make them. In the course of experience, however, there have come to the front elements of cost which had previously been rarely considered. One of these elements is drainage. There are some projects where surface conditions were not deceptive and natural under-drainage has fulfilled all requirements. In some, it is now apparent that artificial drainage should have been provided from the beginning, and there are others where apparently very favorable under-drainage was found blocked by dikes or other obstructions. This experience has led to the recent practice of making provision for drainage cost in all estimates.

Another and far more serious item of cost is the help which, under a new policy of the U. S. Bureau of Reclamation, is to be extended to settlers. When the Reclamation Act was first passed a general storage reservoir in aid of private irrigation, sometimes with and sometimes without a main canal covering a definite area of land, was deemed to constitute all the Federal aid wanted. Subsequently, Federal construction of main and sub-laterals together with drainage were added, and the new policy now contemplates

the construction of laterals on the farm itself and the preparation of the land for irrigation, with the possible further help of financing house, barn, and fence construction, farm implement purchase, and stocking of farms. Additional individual financing is to be done subject to a small rate of interest.

The writer deems the discussion of this subject beyond the scope of this paper. It is merely desired to point out that financial help thus extended from whatever source may require an expenditure of $50 to more than $100 per acre in addition to construction cost and that any financing beyond the conservative extent to which Federal Land Banks have carried such loans, no matter how beneficially it may work out in some cases, is quite certain to involve the Government if it grants such assistance, in direct financial losses, which are essentially a part of the general commitment and which in some manner should be appraised and included in the estimated cost.

There is still another item of cost which has been ignored, sometimes intentionally, in making propaganda for desired projects. Reference is had to the equivalent of what in public utilities is called "early losses."

It has been found that the rate of land settlement in any one locality has a distinct limitation. It depends on the available supply of willing and well-equipped settlers attracted by the local climatic and other conditions. With the rising standard of living much has to be created in the way of roads, schools, banking facilities, domestic water supply, and numerous other things, many of which were only a short time ago regarded as luxuries by the pioneer farmer. Time and money are required to create these things and during their absence settlement is slow and discouraging. Moreover, with the high-priced projects of the future the speculative attraction of a rapid rise in land values, which has been one of the strongest motives where a rapid influx of population took place, will be largely absent. Thus, it will happen that irrigation projects comprising hundreds of thousands of acres which are not capable of being developed in small units, will be faced with an annual operation and maintenance charge far beyond the ability of the earlier settlers to pay.

This condition has existed to a serious extent on some large projects which have been faced with heavy operation and maintenance loss after ten years or more of operation. Yet these projects are small in comparison with some developments now being considered. The practice in Federal irrigation in the past has been to add these losses to construction charges on land yet awaiting settlement and to give this land longer time to pay, always without any addition for interest. Later, some of these charges have been entirely written off as loss. These losses fall of necessity for the time being on the National Treasury, while the loss of interest in any case is a permanent loss to the Nation.

This subject is not argued as a reason for non-development, but because whatever is done should be based on the fullest possible information of the real total cost likely to be involved. One specific case in hand is the Columbia Basin project comprising the enormous area of 1 750 000 acres, all dependent on one long main canal with a number of intervening reservoirs and tunnels. Part of the division canals as well as the main canal will have a capacity

far in excess of the requirements during early years of development. How long it may be before the farmers on the project will be able to pay the operation and maintenance cost on such an extensive and only partly used system no man is wise enough to foretell definitely because the experiment is quite unprecedented; but that for a long series of years heavy losses are unavoidable is entirely certain. This cost and the means of meeting it should be studied and approximated in some manner, so that authorities who may ultimately approve the construction of this and other large enterprises, will not be deceived as to the total subsidy which may be involved.

To summarize the following conclusions are believed to be justified:

(*a*) Increase of irrigated area will and should be slow when and where crop prices do not render farming profitable.

(*b*) Private enterprise will continue to bring about healthy irrigation expansion on a small scale in many localities as soon as crop prices become generally profitable.

(*c*) Irrigation districts will continue to play a large part in the financing of improvements of existing projects and, to some extent, in the construction of small new projects under especially favored conditions.

(*d*) The irrigation district method is not well adapted to the reclamation of large areas of desert land.

(*e*) Continued growth of the West will require the construction of large irrigation projects for the reclamation of desert land.

(*f*) In general, the construction of large projects involving the reclamation of desert land carries under present conditions risk out of proportion to expectation of profits and will not be undertaken by unaided private enterprises for a long time.

(*g*) Stagnation of irrigation development and the consequent slowing down of Western growth may prove of serious detriment to the Nation.

(*h*) State and National subsidy to irrigation is justifiable and desirable, provided it is restricted to the needs of the immediate future and be conditioned on the finding of a reasonable relation of anticipated benefits to total cost, including all items of present and future expenditure.

(*i*) It is important that efforts be made to sever politics from business in the administration of Governmental subsidy.

STATE RECLAMATION IN WASHINGTON*

By R. K. Tiffany,† M. Am. Soc. C. E.

That for many years the State of Washington has recognized the interest of the public in reclamation development is evidenced by many laws designed to aid and promote such development, and more particularly during recent years by the Reclamation and Land Settlement Acts of 1919, the Reclamation Bond Certification Act of 1923, and by the activity of the State in investigating and promoting the development of the great Columbia Basin irrigation project.

Reclamation, as the term is generally used, includes development for agricultural use of swamp, overflow, cut-over, and arid lands. Because irrigation is, and perhaps will continue to be, the predominant phase, this discussion will have chiefly to do with irrigation.

In irrigation development, Washington has passed through the various stages of evolution common to the Western States. First, the little private ditch, the partnership and community ditch, in both of which the development cost was low and the maintenance cost generally negligible. Later, came the stock companies taking up more ambitious enterprises, but the cost under these was generally not more than $10 to $20 per acre and the maintenance cost correspondingly low.

Thus far, all development was by the land owners themselves and without expectation of profit from the irrigation plan, as such. In the Nineties came the promoters, financed by Eastern capital, attracted by a paper-showing of tremendous profits on the investment in irrigation works.

Following these projects came the first serious disappointments. The panic of 1893 checked the stream of westward immigration, thousands of acres of land under ditch lay idle, and operating costs on the small areas under cultivation, together with interest on the large investments in land and canal systems, soon brought financial ruin to practically all these enterprises. In later years, most of them were re-financed and have become rich and prosperous districts, but the initial investments were largely lost.

The topography of the irrigable lands and the river valleys of Washington differs from that of most of the Western States. The streams flow in deep, narrow valleys so that the building of the larger irrigation systems involves expensive construction in the way of tunnels, bench flumes, and pressure pipes. The large bodies of land irrigated and irrigable, with the single exception of the Yakima Indian Reservation, lie on benches at considerable elevations above the river valleys and are generally rolling and steep as compared

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with the plains of Idaho or the gentle slopes of the Salt River and Sacramento Valleys, and most of the other larger irrigated areas of the West. To overcome this combination of physical difficulties means necessarily high cost for the irrigation works. Of the remaining projects in the State of Washington on which careful estimates have been made the construction cost, exclusive of storage, will range from $100 to $150 per acre.

It was the experience of private corporations in the early development and the high cost of all remaining projects, which brought the realization that there is little chance of additional irrigation development in Washington by private capital; and at the close of the World War, with good prices generally prevailing for farm products, with the good lands under existing ditches practically all producing, and with apparently a steady increase in demand for irrigated land, Washington, in 1919, enacted a State Reclamation Law. The object of the Act is declared to be,

"To provide for the reclamation and development of such of the arid, swamp, overflow, and logged-off lands in the State of Washington as shall be determined to be suitable and economically available for reclamation and development as agricultural lands, and the State of Washington in the exercise of its sovereign and police powers declares the reclamation of such lands to be a State purpose and necessary to the public health, safety, and welfare of its people."

A State Reclamation Board was created consisting of the State Commissioner of Public Lands, the State Treasurer, the State Hydraulic Engineer, the State Commissioner of Agriculture, and the President of the State College. This Board was afterward replaced under the new administrative code by the State Department of Conservation and Development. A State Reclamation Service was authorized and a Reclamation Revolving Fund was created by the levy of a tax of 0.5 mill on all taxable property in the State.

The plan of operation provided that the engineer of the State Reclamation Service should investigate projects brought to its attention and that, if found feasible, the Reclamation Fund should be used to purchase the bonds of properly organized irrigation districts, the construction of the works to be under the supervision of the Department, the bonds to be resold whenever they should become salable in the general market.

In the six years following the passage of the law the State Reclamation Service was extremely active, first, investigating new projects, and, later, when it appeared that there were very few projects that could safely assume the burden of repayment of the estimated cost, the activities were largely transferred to investigation and aid of old projects which were in need of additional funds for rehabilitation or extension work.

All told, about forty irrigation, drainage, and diking projects have been investigated. The State has purchased the bonds of sixteen projects to the amount of approximately $1 462 000. Of this amount the bonds of ten districts, aggregating $516 000, have been resold, practically at cost, no addition being made to cover administrative expense. Bonds of eleven districts to the amount of $946 000 are still owned by the State. Of these, $425 500 in

value are considered as probably good, \$316 500 as doubtful, and about \$204 000 as almost certainly a total loss.

In addition to the actual purchase of bonds, advances have been made by the State to various districts for investigation and construction expense which, up to September 30, 1925, amounted to \$840 000. These advances, for the most part, were made under contracts with the districts for repayment in bonds. The two principal items of these advances are \$73 160 for investigation of the Methow-Okanogan Reclamation District, the repayment of which is doubtful on account of the high estimated cost; and \$741 000 for the investigation and construction of the Whitestone Reclamation District, of which at least \$500 000 must be charged off on account of insufficient water supply. The total loss during the first six years' operation of the State Reclamation Service on account of investigations and construction is likely to be between \$777 000 and \$1 093 500—this from a total investment of \$2 200 000, or at the rate of 35 to 49 per cent.

Of the twenty-two projects which were fully investigated and partly or wholly financed by the State Reclamation Fund, five were aided in investigation only. Of the remaining seventeen projects, five only were new projects, the other twelve involving rehabilitation or extension of old privately owned systems. It is obvious that some of these projects were approved rather from a sympathetic motive to settlers on the ground than from any consideration of merit. Some of the heaviest losses may be traced to this cause.

The heaviest single loss is on a project which had been developed privately to serve about 800 acres and which was projected to irrigate more than 8 000 acres at a total estimated cost, including two years' interest, of \$900 000. On this project, reports of competent engineers, calling attention to the inadequacy of the water supply, were completely ignored, the project was laid out and about 90% completed on the basis of serving about 8 300 acres. In September, 1925, after a comprehensive study of water supply and of storage possibilities, a board of engineers recommended the reduction of the project to 2 500 acres located in certain definite divisions, and after classification of the lands in these divisions, the District Board of Directors, with the unanimous consent of resident land owners, took action to reduce the area to 2 207 acres of irrigable land.

Two other projects of about 1 000 and 3 000 acres, respectively, are placed in the doubtful class because, although intrinsically good, they are remote from markets and were apparently built in advance of demand for the land. Efforts are now being made by land owners, bond-holders, and various local interests to secure settlers for these projects. If these efforts are reasonably successful within the next year or two, these projects may be placed on a paying basis.

Results of operation under the State Reclamation Law to date have brought a sharp difference of opinion as to the wisdom of the State continuing in the business of reclamation. Those in favor of continuing point to the beneficial results of the State's assistance to many projects at times when it was practically impossible for them to secure funds for essential reconstruction or

additional water supply through the usual banking channels. In passing, it should be said that this aid to "lame ducks", although no doubt beneficial, was not within the intent of the original Act and should not be the controlling factor in determining future State reclamation policy.

Opponents of the idea of State reclamation point out that it is unfair to tax present farmers to provide additional market competition for their products, prices for which in recent years have not been even fairly remunerative. They also point to the heavy losses to the State by reason of its venture in reclamation.

A Special Committee of the State Legislature was appointed at the Special Session in 1926 with instructions to study results of State reclamation to date and to report on or before October 1, 1926, with recommendation as to a future State reclamation policy and program.

The recommendations made by the Committee are as follows:

"1. That the laws with respect to reclamation and land settlement be retained in the statutes of the State of Washington. That the organization of the department be continued for the purposes of study of resources of the State, possibilities of successful reclamation, and for co-operation with the Federal Reclamation Department in the proper development of projects which have been undertaken by the Federal Government within the State. That depository interest and bond interest on moneys in the reclamation fund be conserved within the said fund to replace losses which have accrued and to defray the administration expenses of the department.

"2. That the State department do not undertake the financing of construction on large new projects or projects which have failed in attempted development by private capital.

"3. That the department consider the possibilities of assisting in the internal improvement of proven and successful reclamation projects by investment in the bonds of such districts, and assistance in marketing same at as low a rate of interest as is possible.

"4. That the reclamation fund levy be suspended for a period of two years, as was done at the last session.

"5. That the State co-operate with the Federal Government in affording such assistance as is required by the Federal Government and is within the power of the State to render in and about the construction and development of those reclamation projects definitely approved by the Federal Reclamation Department."

It is of the greatest importance that this problem be considered carefully and without prejudice, having in mind not only the results of past operations under the law, but the probable future needs of the community for reclamation development, and possible alternative means of meeting such need through State, Federal, or private agencies.

It will be conceded by friend and foe alike of State reclamation that the State of Washington eventually will need all possible development of her agricultural area. Her rapid industrial and commercial development during the past few years, with promise of still more rapid growth in the future, based on her native resources in raw materials, her abundant and cheap water power, and her accessibility through maritime commerce to the raw materials and markets of all the countries bordering the Pacific, is sure to demand reasonably steady development of her agricultural possibilities.

Past experience and the magnitude and high cost of the undeveloped reclamation projects in the State will make it difficult, if not impossible, to interest private capital in these enterprises. Under any plan thus far put forth, the interest charges plus the operation and maintenance deficit during the development period will constitute a burden greater than can possibly be borne by the producing area during the earlier years. Investment bankers have learned from bitter experience that this means default in interest payments, hence the impossibility of interesting private capital in this type of development.

It appears that some public agency must assume the responsibility for carrying the larger irrigation projects through their development period. The United States Reclamation Service has been thus carrying projects ever since its inception. The State of Oregon has carried some projects through loans or direct appropriations for payment of bond interest. Is there any means other than State or Federal subsidy of building the remaining large projects?

One plan that has been suggested is that the county or counties that would benefit most directly from the development of a reclamation project should be organized into a district somewhat along the lines of the Miami Conservancy District, which would carry the burden of interest and of operation and maintenance deficit until the project becomes self-supporting.

An underwriting agency of this character might undertake complete development, or it might become responsible for a definite part only, as, for example, the necessary storage works. In some cases benefits other than for irrigation, such as flood control or power, might largely or entirely cover the cost of storage development. Such a plan, although theoretically workable and, in some cases, justifiable by reason of indirect benefits to the underwriting territory, would contain elements of difficulty and danger. If otherwise practicable it should be supported by provisions that would give to the underwriting district complete control of land development in the project so that this development should come in progressive blocks rather than in scattered units throughout the project, based on individual ownership.

This calls for a consideration of some of the fundamental weaknesses of the present irrigation district law as a means of developing large irrigation projects. Like the district laws of most Western States, the Washington law is based on the old "Wright" Act of California, which was undoubtedly framed to meet the needs for financing groups of actual farmers and settlers who were, as a rule, occupying and farming the lands for which irrigation was sought. Under such conditions most of the land would become productive within a short time and might reasonably be expected to bring returns sufficient to meet both interest and other irrigation charges.

In the attempt to apply this law to larger projects, however, difficulties and reverses have developed. Lacking actual settlers ready to make use of the land and water, there has often been a scattering and haphazard development requiring years to bring the project to a position in which operating charges and interest could be met from returns from the land. Scattering, partial development means not only excessive operation and maintenance costs

and heavy interest burden on the producing area, but also excessive costs for roads, schools, telephones, and the necessary commercial and marketing agencies. For all these items, whether public or private, are a part of the service requirements of any new community, and the cost of establishing and maintaining them must be borne ultimately by the producing land. If the productive land is scattered, a higher cost must be paid, not only in higher tax levies for public institutions, but also in higher interest rates at the bank and longer profits to "the butcher, the baker, the candlestick maker".

The Yakima project in the State of Washington has been widely advertised as one of the most successful instances of irrigation development in the West. There are three outstanding reasons for the success of this project: First, its productive soil and favorable climate; second, its ample water supply; and, third, the fact that it has been developed in small and rather compact units so that its growth has been approximately commensurate with the demand for its land and products.

It must be recognized that the large projects remaining in Washington and other States are composed for the most part of strictly arid lands, with few or no *bona fide* settlers. The problem of building the works is simple compared with that of settling the project and bringing it into production. To do this the land must be pooled, by some means, so as to be handled with the water and irrigation works under a single competent, aggressive, and economical management. The present individual ownership must be merged for the common good, with compensation to present owners based on dry land value only. Whether the financing is done through private or public channels, the agency furnishing the funds must have the controlling hand in the operation and management of the district until it is safely established on a paying basis.

Modifications of irrigation district laws along the lines just suggested are desirable, not only to insure the success of new projects, but for the protection of existing development. The unplanned and haphazard settlement and development of an irrigation project of 100 000 to perhaps more than 1 000 000 acres is not only difficult and dangerous to the new project in itself, but may prove seriously detrimental to established agricultural interests by unsettling their markets. For example, the effect of the great irrigation development in Southern Idaho during the decade from 1905 to 1915, may be recalled, when for a time alfalfa, the easiest crop for new settlers, was produced in quantity far above the capacity of available markets. The market was wrecked, and it required years of patient and energetic effort to build up the dairy and livestock industries and to develop new markets to re-establish the economic balance.

Consider some of the obvious defects of the Washington State Reclamation Law. If State aid for reclamation becomes desirable and necessary at some future time, it should be on a basis that will offer to the project real help and not impossible burdens. Any one familiar with the undeveloped projects in the State of Washington will recognize that few, if any, of them can honestly undertake to meet the requirements of the law as it now stands.

It contains at least two basic defects. First, the law does not require specifically, and in its administration little attention has been paid to, any showing of economic need for a project. Direct State aid to reclamation can be justified only on the ground that there is a real need for the particular development proposed, and that there will be a market, present or prospective, for its products. Second, the interest rate is too high. To make the law workable and of benefit to the settlers, interest should be made very low or waived entirely during the development period. After development of the project is well along, the rate should be high enough so that the district bonds could be sold in the open market at or near par value.

Should conditions in the future warrant direct State aid in reclamation development, it might be advisable to re-establish a Reclamation Board somewhat along the lines provided in the original Act, its functions, however, to be largely advisory on matters of policy and on approval of projects, and not administrative. On such an advisory board, representation should be given to the State College, the State Land Commission, the State Departments of Agriculture and of Conservation and Development, the State Treasurer, and, possibly, the State organization of investment bankers. Most of the bonds handled by the State would probably be sold through the membership of this latter organization and its advice, being expert and non-political, should be of great value.

State Land Settlement Law.—The Washington State Land Settlement Law was enacted in 1919 to meet the anticipated passage of a National land settlement law which was at that time under consideration by Congress on recommendation of the then Secretary of the Interior, the late Franklin K. Lane, the measure having been prepared by Elwood Mead, M. Am. Soc. C. E.

The National legislation failed of passage, but in 1921 the State organization of the American Legion urged and secured a State appropriation for soldier settlement, independent of National aid. The project selected was in the south central part of the State in the vicinity of White Bluffs and Hanford, on the Columbia River. It was doomed to failure from the beginning for several reasons, among which may be mentioned:

1.—The land was mostly very sandy and difficult to place in production. Only irrigation farmers of long experience under similar conditions could expect to achieve success.

2.—Settlement was at first limited to *ex*-service men which meant, generally, young men inexperienced in handling soils of this type.

3.—The water supply from wells was unreliable and in many cases insufficient, and the cost of electric pumping was very high, especially for the development period.

4.—The tracts were scattered, making community life and organization difficult. Community organization and co-operation is one of the essential features of Mr. Mead's plan on which the State law was founded.

5.—The land selected was remote from markets and from towns or cities offering the cultural and recreational facilities to which most of the settlers were accustomed.

The State purchased approximately 2 000 acres of land which was divided into 105 tracts of about 20 acres each. Sixty-six of these tracts were im-

proved with small houses, barns, wells for irrigation, and pumping equipment, and were sold. Thirty-nine other tracts were partly improved and offered for settlement with no takers.

For two to five years the settlers struggled valiantly to establish themselves on the land. Some, with the better tracts of land, with some experience and ample water supply, were able to make a very good showing. Others, due to lack of experience, poor land, or insufficient water, accomplished very little. Nearly all were discouraged and unable to see any possibility of paying out. They appealed to the Legislature for relief, which was authorized. Of the 66 tracts sold, settlements were made with 64 owners at a total net loss to the State of $360 000. The remaining 39 tracts, appraised at $34 000, were sold at public auction on June 1, 1926, with all improvements, except motors and pumps which had been removed, for $50 650.

It cannot be said that this was a fair test of State-aided land settlement. The land was poorly selected, remote from markets, very difficult to reclaim, and the limitation to *ex*-service men developed a certain class feeling and political situation that would be a serious handicap for any project. The results of the experiment however, have brought out several fundamental difficulties in the way of State-aided land settlement, and it is not likely that the State of Washington will be willing to undertake another project of this kind for several years to come.

Reclamation Bond Certification Law.—Following the example of Oregon and California in an effort to improve the marketability of the bonds of sound irrigation districts, the Washington State Legislature in 1923 enacted the so-called "Certification Law". The law requires an investigation by the Director of the Department of Conservation and Development and the filing of a report with the Secretary of State, embracing conclusions on the following points:

(*a*) The supply of water available for the project and the right of the district to so much water as may be needed.

(*b*) The nature of the soil as to its fertility and susceptibility to irrigation, the probable quantity of water needed for its irrigation, and the probable need of drainage.

(*c*) The feasibility of the district's irrigation system and of the specific unit for which the bonds under consideration are desired, whether such system and unit be constructed, projected, or partly completed; and the sufficiency of the amount of the proposed bond issue to complete the improvement contemplated.

(*d*) The reasonable market value of the water, water rights, canals, reservoirs, reservoir sites, and irrigation works owned by such district or to be acquired or constructed by it with the proceeds of any such bonds.

(*e*) The reasonable market value of the lands included within the district.

(*f*) The plan of operation and maintenance used or contemplated by the district.

(*g*) The method of accounting used or proposed to be used by the district.

(*h*) Any other matter material to the investigation.

Attached to the report of the Director shall be the following:

(*a*) A certificate signed by the Supervisor of Hydraulics certifying to the amount and sufficiency of water rights available for the project.

(*b*) A certificate signed by a soil expert of the Washington State College, certifying as to the character of the soil and the classification of the lands in the district.

(*c*) A certificate signed by the Supervisor of Reclamation approving the general feasibility of the system of irrigation.

(*d*) A certificate signed by the Attorney General of the State of Washington approving the legality of the organization and establishment of the district and the legality of the bond issue offered for certification.

On satisfactory showing as provided, the Secretary of State signs a certificate in the following form which is attached to all bonds issued by the district:

"Olympia, Washington, ... (insert date) ....

"I, ..., Secretary of the State of Washington, do hereby certify that the above named district has been investigated and its project approved by the Department of Conservation and Development of the State of Washington; that the legality of the bond issue of which this bond is one has been approved by the Attorney General of the State of Washington, and that the carrying out of the purposes for which this bond was issued is under the supervision of said department, as provided by law.

"(Seal) ....

"Secretary of State."

Owing presumably to the slump in irrigation securities there have been but few requests for certification under this Act, and the bonds of five districts only have been certified. Of these, three were to cover the cost of reconstruction for old districts and two were for construction of new district works. Since the enactment of the Certification Law, investment bankers have rather insisted on certification, and the few bonds sold without it have suffered heavy discounts.

Columbia Basin Project.—The State of Washington, not content with extending aid of a general character to reclamation enterprise through the medium of the legislation referred to, has seen fit to adopt, as a State project, and by appropriation of State funds to investigate and promote plans for the development of an irrigation project equal in extent, in estimated cost, and potential production to all the irrigation projects completed during the first quarter century under the Federal Reclamation Law, namely, the Columbia Basin project.

This project embraces approximately 1 883 000 acres of irrigable land lying in Eastern Washington in the "Big Bend" of the Columbia River from which it takes its name. The land lies between 350 ft. and 1 700 ft. above seal level. In soil and climate it closely resembles the Yakima and Wenatchee Valleys which join it on the west.

Two principal sources of water supply have been considered: First, from Clark's Fork of the Columbia River by a gravity diversion at Albany Falls near the Washington-Idaho boundary, with storage supplied by Pend Oreille Lake, or by this lake supplemented by Priest Lake, also in Idaho; and, sec-

ond, by pumping from the Columbia River, using power developed by a dam in that stream to lift the main water supply into Grand Coulee, from the lower end of which it would be delivered into the main canals.

These two alternative projects lend themselves to numerous variations through the use of other sources of water supply for various parts of the main project, as follows:

Wenatchee River and Lake for Quincy unit	332 000	acres
Snake River for Five-Mile Rapids pumping unit	100 000	"
Touchet River and Wynette Reservoir for part of Eureka Flat	50 000	"
Columbia River, Priest Rapids pumping unit	113 000	"
Spokane River and Lake Coeur d'Alene, southeastern part of gravity project, about	400 000	"

This project, first investigated by the U. S. Reclamation Service in 1903, was then abandoned as being too large for present consideration. In 1919, the Legislature of the State of Washington appropriated $100 000 for its investigation and created a Columbia Basin Commission which, later, became a Division of the Department of Conservation and Development. This was followed by other State and Federal appropriations under which extensive preliminary surveys and studies have been made. The latest and presumably the most authoritative report was made in 1925 by the following Board of Engineers appointed by the Secretary of the Interior: Messrs. Louis C. Hill, Joseph Jacobs, Charles H. Locher, Richard R. Lyman, Arthur J. Turner, and O. L. Waller, all of whom are members of the Society.

This Board finds that the project is structurally and economically feasible and includes these rather striking recommendations as to State and National policy in connection with the proposed development:

"That the State should assume its proper share of the responsibility for collecting payments from the settlers, and should also bear its proper share of the losses, if any, incident to the development of the project."

* * * * * * * *

"That the Government should clear and level the land and provide a reasonable financial credit for necessary farm improvements. Also, as a guarantee against land speculation, and to insure that the settler secures the land at its fair value, that the Government acquire title to all the irrigable land within the project."

The estimated construction cost of the project as a whole, or of its constituent units, is approximately $160 per acre. If the suggestions of the Board of Engineers relative to land ownership can be sucessfully carried out, the combined cost of land and water to the settler might be approximately $175 per acre, which is considerably less than that on the later developed units of the Yakima project and some of the other more successful private and Government projects of recent years.

It is recognized that because of its magnitude this development can be undertaken only by the Federal Government. The State at present is only laying the foundation for future construction by negotiating a water com-

pact with other States interested and with the Federal Government by making an economic survey as to the need and possible market outlet for its products, and a further study of potential power in connection with, or which might be affected by, the irrigation project. The economic survey will include a study of the rate of industrial and commercial growth of the western part of the United States; domestic and foreign markets for agricultural products; past and present production of irrigated and other crops in the Pacific Northwest; and possible new products and new markets which might become available through the opening of new arteries of commerce or reduced tariffs over existing lines.

The conclusions, therefore, as to the proper functions of the State with reference to reclamation, which are briefly summarized, are as follows:

1.—Present agricultural production in this and neighboring States is equal to, if not greater than, the demand in markets that can be economically reached through available channels. The State, therefore, is not justified in expending funds raised by a tax on all property, including agricultural land and products, to increase the agricultural area by reclamation. Having in mind that the rapid industrial and commercial development of the Pacific Coast may, in a relatively short period, radically change the economic situation, the State should anticipate future needs for reclamation development by adopting a sound, basic reclamation policy. The program, as far as it can be outlined, should include:

> (*a*) A comprehensive study of the possibilities and needs of the State for increased agricultural production by means of reclamation. The water supply required for approved projects should be protected by administrative or legislative withdrawal against appropriation for less essential uses.
>
> (*b*) A continued effort to improve State laws relating to reclamation to the end that they may encourage and foster sound and necessary projects and prevent the launching of those which are not.

2.—The State should support a National reclamation policy and program based on sound, economic principles. Selection of projects on merit only; emphasis on local responsibility, especially in operation of the projects; and a strictly business corporation for collection of charges, are some of the factors that would seem to be clearly indicated by past experience as essential to future success. During periods, such as the past few years, when agriculture is in distress and their is little or no demand for land, storage construction should be stressed rather than the building of additional canals, in order to provide for and expedite the development of new areas when needed.

3.—A study of existing projects that are now in trouble and the working out of salvage plans where practicable might be justified, particularly on those projects in which the State already has some investment or has been responsible in some measure for existing conditions.

THE DESIGN OF A MULTIPLE-ARCH SYSTEM AND PERMISSIBLE SIMPLIFICATIONS

Discussion*

By A. C. Janni,† M. Am. Soc. C. E.

A. C. Janni,‡ M. Am. Soc. C. E. (by letter).§—The writer at once wishes to call attention to a statement which, to do full justice to all, should have been incorporated in the original paper.|| The statement was made at the time of presentation,¶ as follows:

"The method followed [in the paper] is that of Professor Guidi of the Polytechnique of Turin. This material I have taken from his work of four or five volumes.** Incidentally, I would say that it is too bad that this work has not as yet been translated into English; it would be a great help to the Engineering Profession".

In view of certain remarks made by discussors, concerning some points of this paper, the writer has found it expedient to elucidate further on these points by extending his research so as to make more clear the legitimacy of this simplification.

Assume that Arch A_2 is the center arch of a five-arch system, A_0, A_1, A_2, A_3, A_4. For the sake of brevity assume that these five arches and their four piers, P_0, P_1, P_2, P_3, are of the same size as those shown in the case of a three-arch system.

Determination of the Ellipses of Elasticity of the Various Combinations of the Parts of This System

The ellipse of elasticity, G_{A_0}, of Arch A_0, is, of course, already known, also the ellipse of elasticity, G_{P_0}, of Pier P_0, as well as the ellipse of elasticity of the joint, $G_{A_0P_0}$, between Arch A_0 and Pier P_0.

The determination of the ellipse, $G_{A_0P_0A_1}$(combination of the two ellipses, $G_{A_0}P_0$ and G_{A_1}) is conducted as follows:

Let, I_x = moment of inertia with respect to the horizontal diameter, xx.
I_y = moment of inertia with respect to the vertical diameter, yy.
I_{xy} = product of inertia with respect to the diameters, xx and yy.
α = angle of inclination of the major axis to the horizontal.

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* Discussion of the paper by A. C. Janni, M. Am. Soc. C. E., continued from *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1244.

† Author's closure.

‡ Cons. Engr., New York, N. Y.

§ Received by the Secretary, July 19, 1926.

|| *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1142.

¶ At the meeting of the Society, September 3, 1924.

** "Lezioni sulla Scienza delle Costruzioni," by Professor Camillo Guidi, Torino, Italy.

I_a = moment of inertia with respect to the minor axis.
I_b = moment of inertia with respect to the major axis.
ρ_a = length of major axis.
ρ_b = length of minor axis.

Then,

$$I_x = 373.54 \times (1.30)^2 + 15.55 \times (6.17)^2 + 373.54 \times (0.05)^2 + 15.55 \times (1.22)^2 = 1247.05$$

$$I_y = 373.54 \times (12.90)^2 + 373.54 \times (1.02)^2 + 15.55 \times (1.09)^2 + 15.55 \times (24.65)^2 = 72022.01$$

$$I_{xy} = 1.22 \times 24.65 \times 15.55 + 1.02 \times 0.05 \times 373.54 = 491.67$$

$$\tan 2\,\alpha = -\frac{2 \times I_{xy}}{I_x - I_y} = 0.01389\,;\ 2\,\alpha = 0^\circ\ 48'$$

$$I_a = \frac{1247.05 + 72022.01}{0.2} + \frac{1}{2}\sqrt{(70774.96)^2 + 4 \times (491.67)^2} = 72\ 025$$

$$I_b = \frac{1247.05 + 72022.01}{0.2} - \frac{1}{2}\sqrt{(70774.96)^2 + 4 \times (491.67)^2} = 1\ 243$$

$$\rho_a = \sqrt{\frac{72025.}{389.09}} = 13.61 \text{ ft.}$$

$$\rho_b = \sqrt{\frac{1243.}{389.09}} = 1.78 \text{ ft.}$$

Fig. 64 shows the graphical method used for the same determination (see Appendix, Paragraphs 8 and 9*). Naturally, the results are the same for both methods. In Fig. 64 is shown the relative position of the ellipse, $G_{A_0P_0A_1}$, with respect to the corresponding ellipse, G_{A_1} and $G_{A_0P_0}$.

It is to be observed that, unless the two elastic centers, $G_{A_0P_0}$ and G_{A_1}, are on the same horizontal line, the axes of the resulting ellipse, $G_{A_0P_0A_1}$, cannot be one horizontal and the other vertical; but they will be inclined to these directions by a certain angle, which, in this case, is 0° 24′. A glance at the expression for the value, d_1,† will show that this exceptional position of the two centers can never be verified.

In this case, however, as the angle made by the major axis of the resulting ellipse with the horizontal is very small, it may be assumed to be zero without appreciable error in the results, and, in so doing, all complications in the next determination of the ellipse of the joint between $G_{A_0P_0A_1}$ and G_{P_1} resulting from this inclination of the axes may be avoided.

The determination of this last ellipse, $G_{A_0P_0A_1P_1}$, is made in the same way as the determination of the similar ellipse of the preceding joint. The result is shown in Fig. 65, the values referred to the ellipse being:

$$d_1 = 16.80 \text{ ft; } \rho = 7.65 \text{ ft; } G_{A_0P_0A_1P_1} = 16.20$$
$$d_1' = 0.006 \text{ ft; } \rho' = 1.07 \text{ ft.}$$

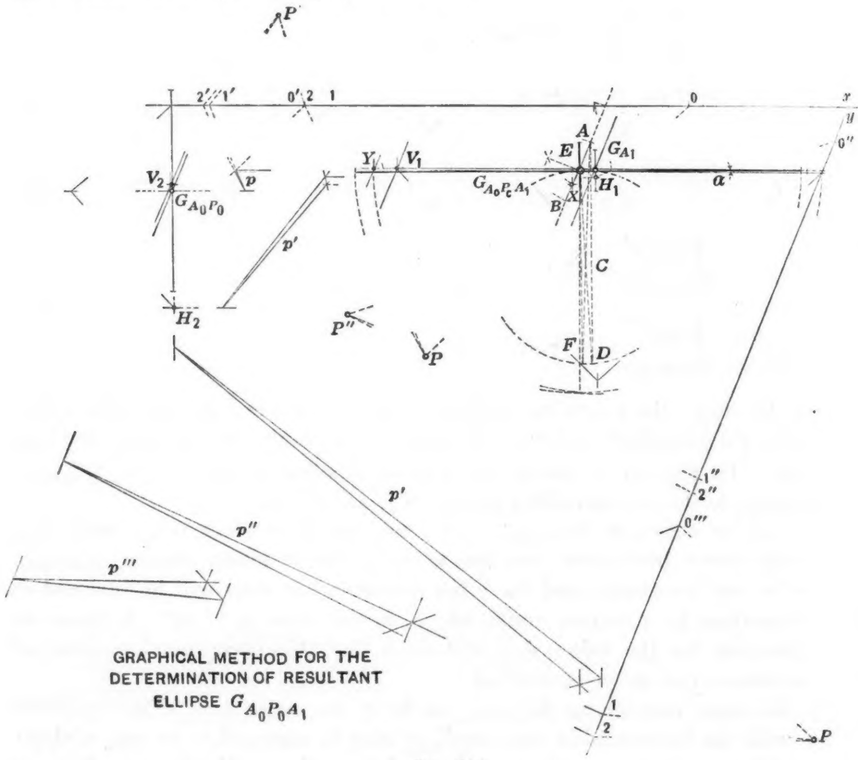
* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), pp. 1164–1165.

† *Loc. cit.*, p. 1152.

Analysis of Arch A_2 (Center Arch of a System of Five Arches)

Unyielding Foundations.—The analysis of Arch A_2 is carried out exactly as in the paper (in which A_2 was the central arch of a three-arch system), the only difference being that in the present instance, Ellipse $G_{A_0P_0A_1P_1}$ takes the place of Ellipse $G_{A_1P_1}$.

Incidentally, in the drawing of the five diagrams (Polygons p_1, p_2, p_3, p_4, and p_5), there is a simplification, which the writer would suggest, and which may be applied in case this method of design is used. A certain arrangement of computations was suggested in the paper.* Similar computations for the present Arch A_2 are as given in Table 12.



GRAPHICAL METHOD FOR THE DETERMINATION OF RESULTANT ELLIPSE $G_{A_0P_0A_1}$

Fig. 64.

From Table 12, the following results are obtained,

$$\lambda_3 = 10.00 \text{ in.};\ \zeta\, \Sigma\, w = \frac{40.58}{7} = 5.79$$

$$\rho = \sqrt{\frac{1}{7} \times 5.07 \times 2.18} = 1.19 \text{ in.}$$

$$\rho_1 = \sqrt{\frac{5.07 \times 10.00 \times 40.58}{40.58}} = 7.12 \text{ in.}$$

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1146.

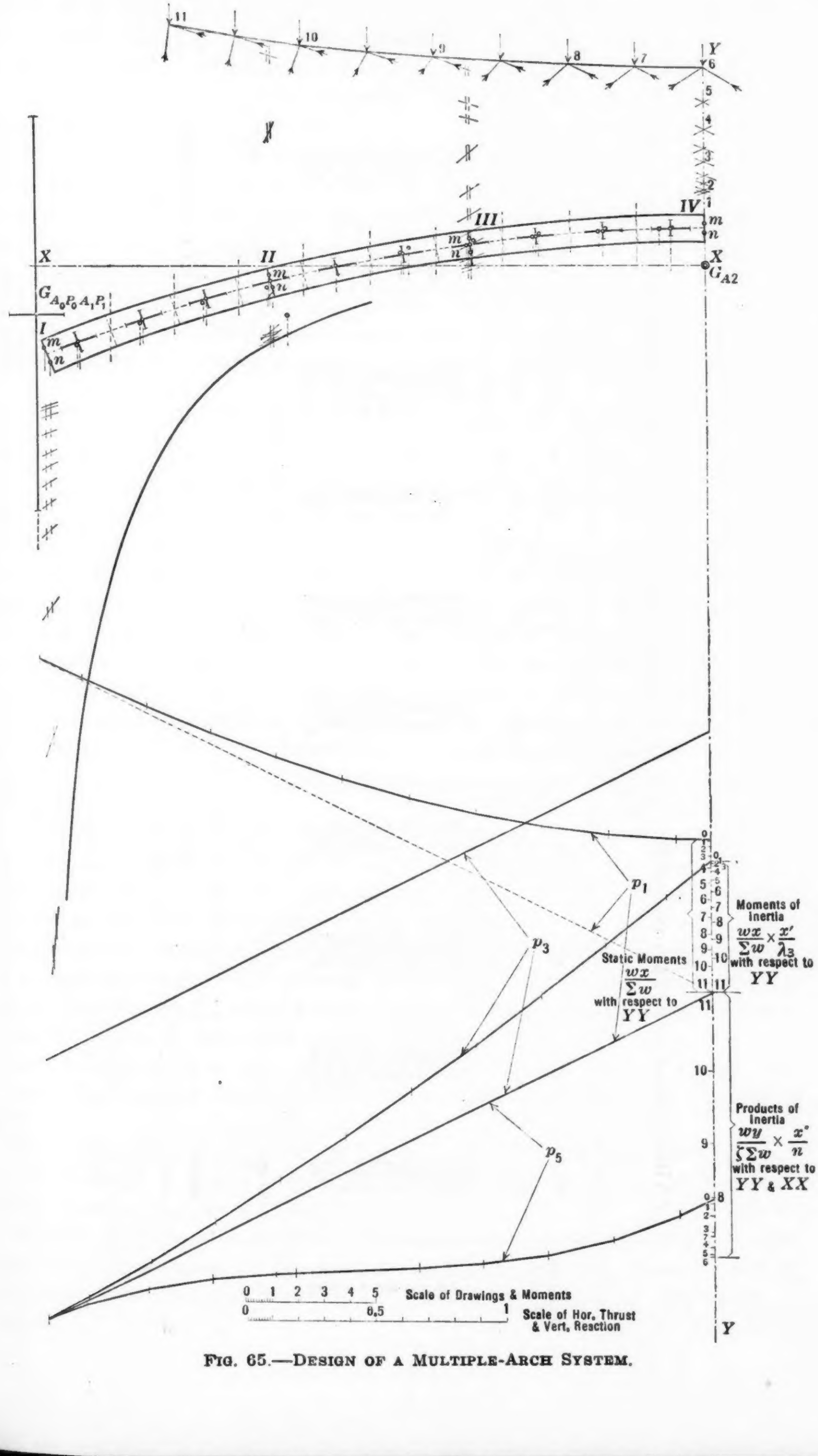


FIG. 65.—DESIGN OF A MULTIPLE-ARCH SYSTEM.

TABLE 12.—COMPUTATIONS OF ELASTIC QUANTITIES FOR CENTER ARCH A_2, OF FIVE-ARCH SYSTEM, UNYIELDING FOUNDATIONS.

Position of load.	w. (1)	x. (2)	y. (3)	x'. (4)	y'. (5)	x''. (6)	$\frac{w\,x}{\Sigma\,w}$. (7)	$\frac{w\,y}{\zeta\,\Sigma\,w}$. (8)	$\frac{w\,x}{\Sigma\,w} \times \frac{x'}{\lambda_3}$. (9)	$\frac{w\,y}{\zeta\,\Sigma\,w} \times \frac{y'}{v}$. (10)	$\frac{w\,y}{\zeta\,\Sigma\,w} \times \frac{x''}{n}$. (11)
1	2.55	0.64	0.71	0.85	0.73	0.63	0.04	0.31	0.008	0.04	0.08
2	2.41	1.94	0.66	2.08	0.70	1.93	0.11	0.27	0.022	0.03	0.23
3	2.28	3.23	0.56	3.29	0.60	3.20	0.18	0.22	0.059	0.02	0.32
4	2.15	4.51	0.41	4.56	0.48	4.44	0.23	0.15	0.104	0.01	0.30
5	2.03	5.79	0.21	5.84	0.34	5.66	0.29	0.07	0.169	0.004	0.18
6	1.93	7.06	−0.03	7.10	− 1.00	8.02	0.33	−0.01	0.234	0.001	−0.03
7	1.72	8.31	−0.33	8.31	− 0.43	8.40	0.35	−0.10	0.290	0.007	−0.38
8	1.38	9.57	−0.66	9.57	− 0.74	9.60	0.32	−0.16	0.306	0.02	−0.70
9	1.19	10.81	−1.06	10.81	− 1.11	10.83	0.31	−0.22	0.335	0.04	−1.09
10	1.03	12.03	−1.50	12.03	− 1.54	12.04	0.30	−0.26	0.360	0.07	−1.43
11	1.62	12.83	−0.94	12.86	−16.78	12.83	0.51	−0.26	0.655	0.86	−1.53
....	20.29 2	......	......	......	......	......	......	......	2.537 2	1.091 2	......
....	40.58	......	......	......	......	......	......	......	v = 5.074 in.	n = 2.182 in.	......

It is to be noted that, except for Columns (2), (3), (4), (5), and (6), which contain distances taken directly from Fig. 65, the remaining Columns (7), (8), (9), (10), and (11), are computations made by the slide-rule and represent: Column (7), the static moments (intercepts on $Y\,Y$) of the various elastic weights with respect to $Y\,Y$; Column (8), the static moments (intercepts on $X\,X$) of the same elastic weights with respect to $X\,X$; Column (9), the moments of inertia (intercepts on $Y\,Y$) of the same elastic weights with respect to $Y\,Y$; Column (10), the moments of inertia (intercepts on $X\,X$) of the same quantities with respect to $X\,X$; and, finally, Column (11), the products of inertia of the same quantities with respect to $X\,X$ and $Y\,Y$.

Thus it is clear that the drawing of Polygons p_2 and p_4 (Columns (8) and (10)), which was done for demonstration, may be omitted in practical cases in which usually the assumed loading is vertical. After all, Polygons p_2 and p_4 are drawn solely to obtain the quantities, $\frac{w\,y}{\zeta\,\Sigma\,w}$ and n, which are needed to draw Polygon p_5; these quantities can be had from Columns (8) and (10) as shown. The position of the elastic center of the arch can be determined by simple computations.

Furthermore, there are other simplifications. Polygons p_1, p_3, and p_5 (Columns (7), (9), and (11)), can be drawn without drawing first the relative force polygons. In fact, Column (7) gives the static moments of w with respect to $Y\,Y$. The corresponding lengths can be laid off on the axis, $Y\,Y$, as shown on Fig. 65 and the various sides of Polygon p_1 can then be drawn. Similarly, the two other polygons, p_3 and p_5, can be drawn by joining points with lines, rather than by constructing parallels to the radii of the force polygons, P_3 and P_5.

Fig. 65 shows this simplified and shortened method for obtaining the three diagrams, p_1, p_3, and p_5. These simplifications reduce the graphical work considerably.

The writer will not explain again the method followed for obtaining the influence and intersection lines, for it is exactly the same as shown in the paper; neither will he explain again the method for obtaining Table 13, which gives the moment for Sections I, II, III, and IV on the hypothesis that Arch A_2 is the center arch of a system of five arches. By following the same reasoning as that for a three-arch system, the forces acting on Arches A_1 and A_0 and on Piers P_1 and P_0, when Arch A_2 is loaded, have been determined (Tables 14 and 15). Fig. 66 shows the results obtained.

It is worthy of note that, while the forces acting on Pier P_1 are of compression, those acting on Pier P_0 are of tension, that is, at Pier P_0 there is uplift.

Therefore, the horizontal thrust for Arch A_0 is larger than that for Arch A_1. This result is shown practically by comparing the values of horizontal thrusts recorded in Tables 14 and 15.

Analysis of Five-Arch System.—Yielding Foundations.—Assume now that, having the same system of five arches, the footings of both Piers P_1 and P_2 yield; the effect that this yielding will have on all arches of this system is required.

TABLE 13.—COMPUTATIONS FOR MOMENTS IN ARCH A_2 (CENTER SPAN OF FIVE-SPAN SYSTEM), WITH UNYIELDING FOUNDATIONS.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.108	− 2.25	− 1.80	−0.243	−0.194	−2.20	−1.80	−0.237	−0.194	−1.19	−0.80	−0.128	−0.086	+1.17	+1.52	+0.126	+0.164
2	0.320	− 2.55	− 2.07	−0.816	−0.664	−2.30	−1.86	−0.736	−0.595	−1.05	−0.70	−0.336	−0.224	+1.50	+1.85	+0.480	+0.592
3	0.547	− 3.20	− 2.72	−1.750	−1.490	−2.42	−2.00	−1.326	−1.094	−0.77	−0.40	−0.423	−0.218	+2.35	+2.70	+1.285	+1.476
4	0.729	− 4.00	− 3.50	−2.916	−2.551	−2.55	−2.07	−1.858	−1.512	−0.23	+0.14	−0.167	+0.102	+3.60	+3.95	+2.624	+2.879
5	0.783	− 4.55	− 4.05	−3.562	−3.171	−2.56	−2.06	−2.004	−1.612	+0.22	+0.65	+0.215	+0.508	+4.65	+5.00	+3.640	+3.915
6	0.798	− 5.20	− 4.70	−4.149	−3.750	−2.55	−2.05	−2.034	−1.635	+0.92	+1.30	+0.734	+1.037	+6.05	+6.40	+4.827	+5.107
7	0.783	− 6.05	− 5.50	−4.737	−4.306	−2.40	−1.92	−1.879	−1.503	+1.85	+2.25	+1.448	+1.761	......	......	....	....
	0.729	− 7.10	− 6.55	−5.175	−4.774	−2.12	−1.62	−1.545	−1.180	+3.30	+3.70	+2.405	+2.697	......	......	....	....
8	0.603	....	....	....	....	......	......	....	....	.....	+7.32	....	+4.413	......	......	....	....
	0.601	....	....	....	....	......	......	....	....	+6.95	......	+4.176	....	......	......	....	....
9	0.547	−10.15	− 9.50	−5.552	−5.196	−0.57	−0.07	−0.311	−0.038	+6.55	+6.85	+3.582	+3.746	......	......	....	....
	0.320	−15.35	−14.50	−4.912	−4.640	+5.45	+6.12	+1.744	+1.958	+5.25	+5.55	+1.680	+1.776	......	......	....	....
10	0.266	....	....	....	....	......	+9.45	....	+2.513	......	......	....	....	......	......	......	....
	0.258	....	....	....	....	+9.07	......	+2.340	....	......	......	....	....	......	......	....	....
11	0.108	−23.62	−21.95	−3.472	−3.284	+8.55	+8.90	+0.923	+0.961	+4.67	+5.00	+0.504	+0.540	......	......	....	....

TABLE 14.—COMPUTATIONS FOR MOMENTS IN ARCH A_1 OF FIVE-SPAN SYSTEM, UNYIELDING FOUNDATIONS.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.087	+ 3.96	+ 4.37	+0.344	+0.880	+ 0.87	+ 1.25	+0.075	+0.108	−0.85	−0.50	−0.073	−0.043	− 1.75	− 1.42	−0.152	−0.123
2	0.238	+ 4.00	+ 4.41	+0.952	+1.049	+ 0.89	+ 1.27	+0.211	+0.302	−0.84	−0.49	−0.199	−0.116	− 1.77	− 1.44	−0.421	−0.342
3	0.392	+ 4.08	+ 4.49	+1.599	+1.760	+ 0.96	+ 1.31	+0.376	+0.488	−0.83	−0.48	−0.325	−0.188	− 1.79	− 1.46	−0.701	−0.572
4	0.502	+ 4.27	+ 4.69	+2.152	+2.363	+ 1.04	+ 1.40	+0.524	+0.705	−0.82	−0.47	−0.413	−0.236	− 1.87	− 1.54	−0.942	−0.776
5	0.518	+ 4.36	+ 4.77	+2.236	+2.447	+ 1.08	+ 1.46	+0.554	+0.748	−0.81	−0.46	−0.415	−0.235	− 1.93	− 1.60	−0.990	−0.820
6	0.500	+ 4.52	+ 4.92	+2.278	+2.479	+ 1.15	+ 1.52	+0.579	+0.766	−0.80	−0.44	−0.403	−0.221	− 2.00	− 1.67	−1.008	−0.841
7	0.483	+ 4.71	+ 5.12	+2.274	+2.472	+ 1.29	+ 1.63	+0.623	+0.787	−0.78	−0.42	−0.376	−0.202	− 2.09	− 1.76	−1.009	−0.850
8	0.413	+ 5.00	+ 5.40	+2.065	+2.230	+ 1.39	+ 1.75	+0.574	+0.722	−0.75	−0.39	−0.315	−0.163	− 2.23	− 1.90	−0.920	−0.784
9	0.258	+ 6.08	+ 6.48	+1.568	+1.684	+ 1.94	+ 2.29	+0.504	+0.595	−0.71	−0.36	−0.184	−0.093	− 2.74	− 2.41	−0.706	−0.621
10	0.094	+ 9.08	+ 9.44	+0.859	+0.925	+ 3.48	+ 3.81	+0.341	+0.373	−0.52	−0.16	−0.050	−0.015	− 4.09	− 3.76	−0.324	−0.353
11	0.009	+37.44	+37.53	+0.374	+0.375	+17.70	+17.82	+0.177	+0.178	+1.18	+1.44	+0.011	+0.014	−17.22	−16.94	−0.154	−0.152

Position of load.	H.	Section V.				Section VI.				Section VII.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.087	− 1.44	− 1.08	−0.125	−0.093	− 0.18	+ 0.16	−0.015	+0.013	+ 2.30	+ 2.72	+0.200	+0.236
2	0.238	− 1.50	− 1.14	−0.357	−0.271	− 0.29	+ 0.06	−0.069	+0.014	+ 2.16	+ 2.58	+0.514	+0.614
3	0.392	− 1.56	− 1.21	−0.611	−0.474	− 0.41	− 0.05	−0.160	−0.019	+ 2.06	+ 2.44	+0.807	+0.956
4	0.502	− 1.75	− 1.40	−0.882	−0.705	− 0.65	− 0.29	−0.327	−0.146	+ 1.69	+ 2.10	+0.851	+1.058
5	0.513	− 1.85	− 1.50	−0.949	−0.769	− 0.82	− 0.44	−0.420	−0.225	+ 1.50	+ 1.91	+0.769	+0.979
6	0.500	− 2.00	− 1.64	−1.008	−0.826	− 1.02	− 0.65	−0.514	−0.327	+ 1.20	+ 1.60	+0.604	+0.806
7	0.483	− 2.21	− 1.85	−1.067	−0.893	− 1.31	− 0.94	−0.632	−0.454	+ 0.81	+ 1.21	+0.391	+0.584
8	0.413	− 2.48	− 2.12	−1.024	−0.875	− 1.73	− 1.35	−0.714	−0.557	+ 0.25	+ 0.67	+0.103	+0.276
9	0.258	− 3.54	− 3.18	−0.917	−0.820	− 3.25	− 2.94	−0.838	−0.758	− 1.83	− 1.38	−0.472	−0.356
10	0.094	− 6.50	− 6.12	−0.611	−0.575	− 7.50	− 7.12	−0.705	−0.669	− 7.56	− 7.08	−0.710	−0.678
11	0.009	−34.60	−34.18	−0.311	−0.307	−48.31	−47.74	−0.434	−0.429	− 62.78	−62.06	−0.565	−0.558

TABLE 15.—COMPUTATIONS FOR MOMENTS IN ARCH A_0 OF FIVE-SPAN SYSTEM (UNYIELDING FOUNDATIONS).

Position of load.	H.	SECTION I.				SECTION II.				SECTION III.				SECTION IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.086	—3.18	—3.62	—0.273	—0.811	—0.50	—0.87	—0.043	—0.074	—0.87	—0.52	—0.074	—0.044	—1.37	—1.04	—0.117	—0.089
2	0.237	—3.17	—3.61	—0.751	—0.855	—0.50	—0.87	—0.118	—0.206	—0.87	—0.52	—0.206	—0.123	—1.37	—1.04	—0.324	—0.246
3	0.388	—3.16	—3.60	—1.226	—1.396	—0.50	—0.87	—0.194	—0.337	—0.87	—0.52	—0.337	—0.201	—1.36	—1.03	—0.527	—0.399
4	0.502	—3.15	—3.59	—1.581	—1.802	—0.50	—0.86	—0.251	—0.431	—0.88	—0.50	—0.441	—0.266	—1.35	—1.02	—0.677	—0.512
5	0.513	—3.14	—3.58	—1.610	—1.836	—0.50	—0.85	—0.256	—0.436	—0.88	—0.53	—0.451	—0.271	—1.34	—1.01	—0.687	—0.518
6	0.504	—3.13	—3.57	—1.577	—1.799	—0.49	—0.84	—0.246	—0.423	—0.89	—0.54	—0.448	—0.272	—1.33	—1.00	—0.670	—0.504
7	0.489	—3.12	—3.55	—1.525	—1.785	—0.48	—0.83	—0.234	—0.405	—0.90	—0.54	—0.440	—0.264	—1.32	—0.99	—0.645	—0.484
8	0.423	—3.06	—3.48	—1.294	—1.472	—0.43	—0.79	—0.181	—0.334	—0.92	—0.55	—0.389	—0.232	—1.31	—0.98	—0.554	—0.414
9	0.273	—2.90	—3.31	—0.791	—0.903	—0.34	—0.71	—0.092	—0.193	—0.94	—0.56	—0.256	—0.152	—1.27	—0.94	—0.346	—0.256
10	0.108	—2.62	—3.04	—0.282	—0.328	—0.19	—0.56	—0.020	—0.060	—0.96	—0.62	—0.103	—0.066	—1.18	—0.85	—0.127	—0.091
11	0.015	—1.20	—1.65	—0.018	—0.024	—0.54	—0.18	—0.008	—0.002	—1.14	—0.79	—0.017	—0.011	—0.69	—0.36	—0.010	—0.005

Position of load.	H.	SECTION V.				SECTION VI.				SECTION VII.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.086	—0.68	—0.33	—0.058	—0.028	—0.87	—1.25	—0.074	—0.107	—3.75	—4.14	—0.322	—0.356
2	0.237	—0.67	—0.32	—0.158	—0.075	—0.87	—1.25	—0.206	—0.296	—3.75	—4.15	—0.888	—0.983
3	0.388	—0.66	—0.31	—0.256	—0.120	—0.88	—1.25	—0.341	—0.485	—3.76	—4.16	—1.458	—1.614
4	0.502	—0.65	—0.30	—0.326	—0.150	—0.89	—1.26	—0.446	—0.632	—3.77	—4.17	—1.892	—2.093
5	0.513	—0.64	—0.29	—0.328	—0.148	—0.90	—1.27	—0.461	—0.651	—3.78	—4.18	—1.939	—2.144
6	0.504	—0.63	—0.28	—0.317	—0.141	—0.91	—1.28	—0.458	—0.645	—3.79	—4.19	—1.910	—2.111
7	0.489	—0.61	—0.27	—0.298	—0.132	—0.92	—1.29	—0.449	—0.630	—3.80	—4.21	—1.858	—2.058
8	0.423	—0.54	—0.20	—0.228	—0.084	—1.04	—1.37	—0.439	—0.578	—3.93	—4.35	—1.662	—1.840
9	0.273	—0.43	—0.08	—0.117	—0.021	—1.23	—1.58	—0.335	—0.431	—4.20	—4.61	—1.146	—1.258
10	0.108	—0.20	—0.12	—0.021	—0.012	—1.54	—1.92	—0.166	—0.207	—4.64	—5.06	—0.501	—0.546
11	0.015	—0.96	—1.29	—0.014	—0.019	—3.29	—3.64	—0.049	—0.054	—7.04	—7.42	—0.105	—0.111

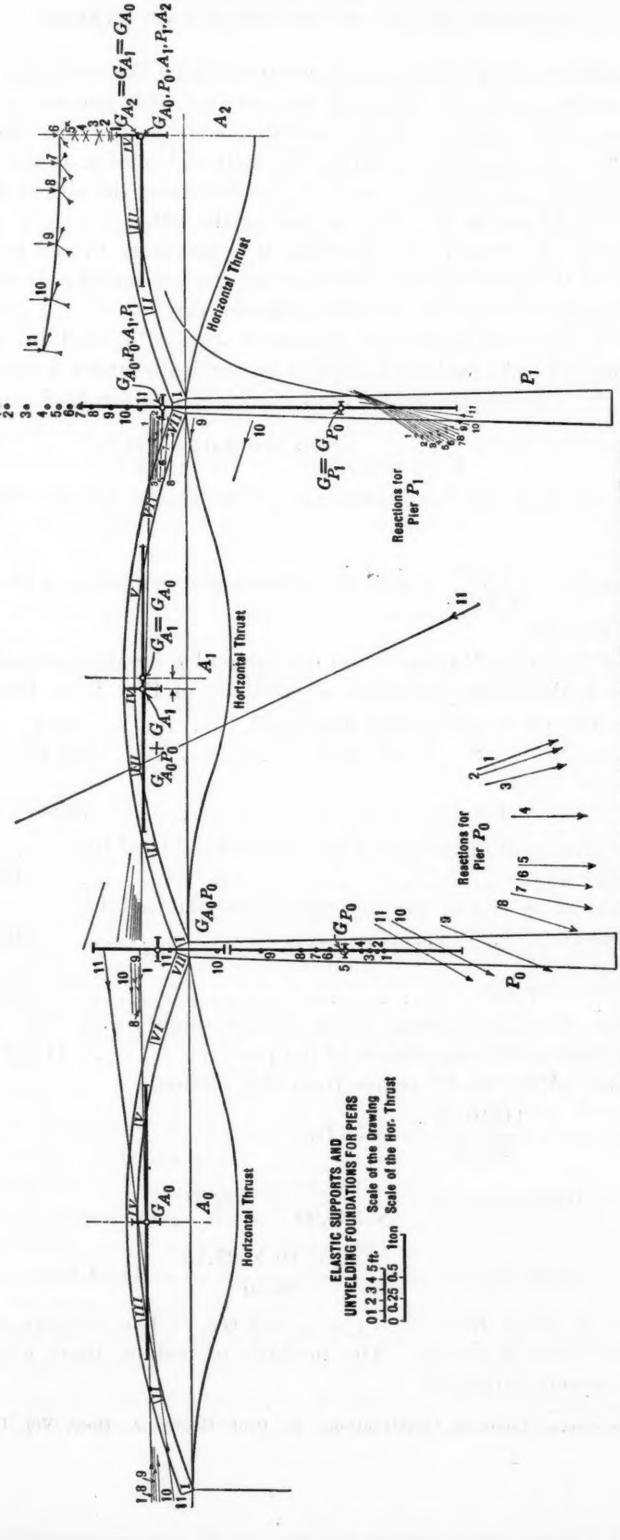


FIG. 66.—ELASTIC SUPPORTS AND UNYIELDING FOUNDATIONS FOR PIERS.

For simplicity the yielding soil is assumed to be the same for both piers. This assumption makes the diagram symmetrical with respect to the center vertical line of the elastic system, reducing considerably the computations involved. For unsymmetrical yielding the method is obviously the same, only longer, for in this case it is necessary also to determine the ellipse of the right joint which would not be the same as that on the left.

In order to take care of this yielding, it is necessary to add to the ellipse of elasticity of the pier another ellipse having the horizontal axis only, for the horizontal displacement of the footing is disregarded.

Having to deal with a bearing surface of 3 ft. 4 in. by 1 ft., assume the nature of the soil to be such as to yield 1 in. vertically under a pressure of 15 tons per sq. ft. The elastic weight of this surface is given by:*

$$\phi_0 = \frac{1}{1\,105\,642} \times 288\,000\,000 = 260.48$$

in which $E = 1$, as has been assumed for the elastic weights of the whole structure.

Therefore, $\rho = \sqrt{\dfrac{3.07}{3.33}} = 0.96$ ft. = horizontal semi-axis of the ellipse of elasticity of the soil.

Ellipse of Pier P_1.—Having found the value of ρ for this particular ellipse, it is possible to determine the ellipse of elasticity of Pier P_1 as follows:

Elastic weight of the pier itself..........	74.61
" " " " soil	260.48
Total...........................	335.09

Sum of moment of inertia of the elastic weights of the pier proper	18.836
Moment of inertia of the soil surface's elastic weight, 260.48×0.96^2..............................	240.058
Total..............................	258.894
Sum of the static moment of the elastic weights with respect to the top section of the pier............	11 510.54

Distance of the elastic center from this section,

$$\frac{11510.54}{335.09} = 34.35 \text{ ft.}$$

$$\text{Minor axis, } \rho' = \sqrt{\frac{258.89}{335.09}} = 0.87 \text{ ft.}$$

$$\text{Major axis, } \rho = \sqrt{\frac{20 \times 10 \times 23.15}{33.50}} = 11.74 \text{ ft.}$$

Ellipse of the Joint Between $G_{A_0 P_0 A_1}$ and G_{P_1}'.—The necessary results for this joint are given herewith. The methods of making these computations follow those already explained.

* "Reinforced Concrete Construction," by Prof. George A. Hool, Vol. III.

$d_1 = 37.78$ ft.; $\rho = 5.98$ ft.; $G'_{A_0 P_0 A_1 P_1} = 25.39$
$d_1' = 0.09$ ft.; $\rho' = 3.14$ ft.

Analysis of Arch A_2.—This analysis is the same in every detail as those previously shown. Fig. 67 gives the three corresponding diagrams. The elastic quantities are given in Table 16. The resulting moments for Sections I, II, III, and IV have been computed and noted in Table 17.

On Fig. 68 are shown the graphical operations necessary for the determination of the forces acting on Arches A_1 and A_0 when Arch A_2 is being loaded and when, furthermore, its supports are built on yielding soil of the assumed modulus of elasticity. The reactions on Piers P_1 and P_0 are also shown. The computations for Arches A_1 and A_0 are given in Tables 18 and 19. A pictorial view of the structures treated in the paper and this discussion is shown in Fig. 69 in a condensed form. The four hypotheses noted are those referred to more in detail in subsequent diagrams.

All the moments obtained in all four hypotheses contemplated and recorded in Figs. 66 and 68 have been plotted together for the sake of comparison. The results (Figs. 70, 71, and 72) show clearly what had been forecast, that the variation of moments for the various sections of Arch A_2 is large only when Arch A_2 passes from fixed supports to elastic supports, constituted each by an arch and a pier, and that the further addition of an arch and pier on each side does not cause any important variation in the moments of the arch.

Interpretation of Results

It is to be noted also that by increasing further the number of arches and piers on each side of Arch A_2, the difference in the moments, between two consecutive increases, will be smaller than the preceding one.

It is interesting also to see the behavior of the system of five arches when the two piers yield according to the assumption made. From an inspection of Figs. 66 and 68, it is seen that the angle formed by Reactions 1 and 11 (extreme reactions) diminishes rapidly.

Considering the various constructions for the determination of the positions of these reactions, it is safe to conclude that in a long series of arches these reactions will tend to crowd together very rapidly, so that there will not be any appreciable error by considering the direction and position of one reaction as that of all reactions for an arch far removed from the center of the system, due to the load of the central arch, and that this direction is very nearly horizontal. In the presence of these results it seems that the simplification proposed by Professor Guidi is fully justified.

In presenting the paper the writer had no idea of giving a complete analysis of a multiple-arch system. His only aim was to emphasize the fact that, having to deal with, say, a system of ten arches in which there are no abutment piers, it is possible to reduce the work of analysis considerably.

There are other structural elements in a system like this, as, for instance, piers, arches, etc., which are subject to proper analysis, whether or not the suggested simplification is adopted for the middle arches.

The writer will outline in a few words the method to be followed in the analysis of the end arches. In order to find the moments, etc., for Arch A_4,

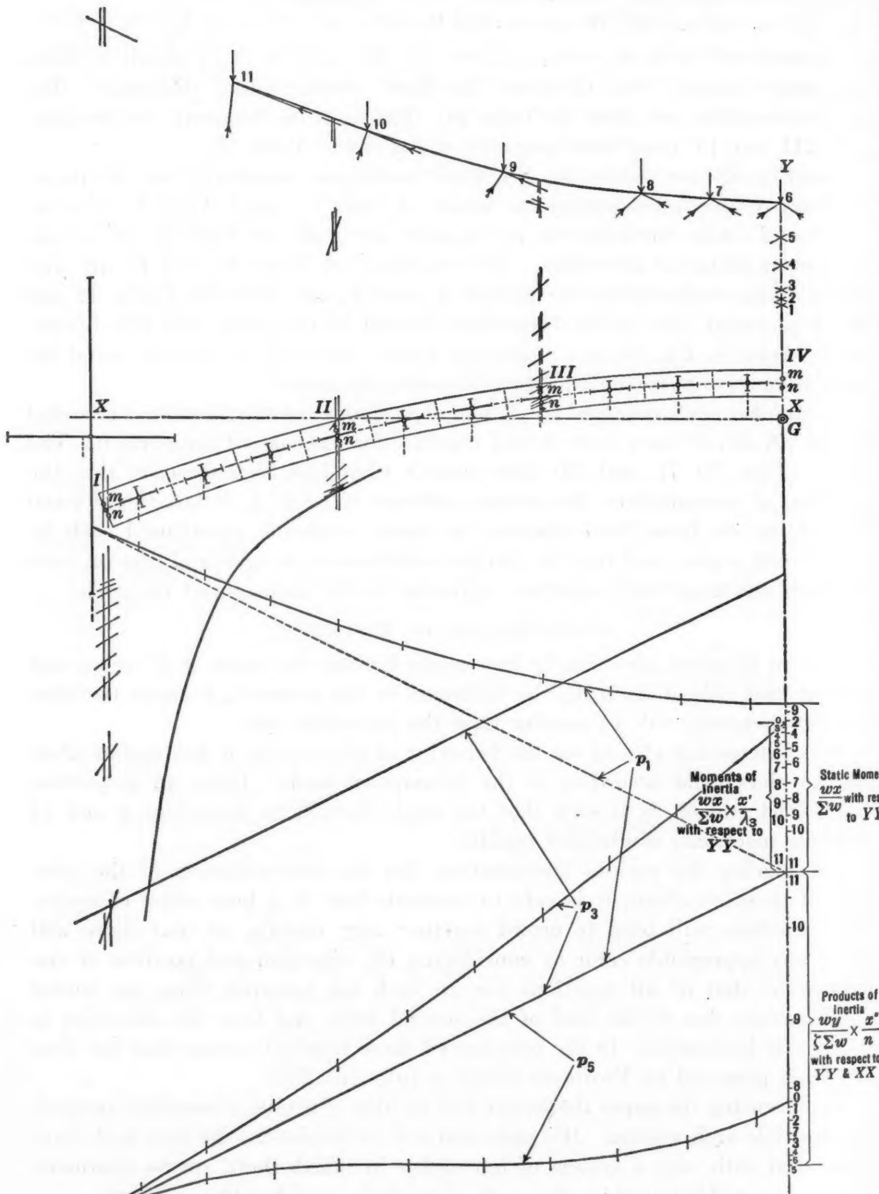


FIG. 67.—DESIGN OF A MULTIPLE-ARCH SYSTEM. ANALYSIS OF ARCH A_2 AS THE CENTER ARCH OF A FIVE-ARCH SYSTEM AND PIERS P_1 AND P_2 BUILT ON YIELDING SOIL.

TABLE 16.—COMPUTATIONS OF ELASTIC QUANTITIES FOR CENTER ARCH A_2 OF FIVE-ARCH SYSTEM, YIELDING FOUNDATIONS.

Position of load.	w. (1)	x. (2)	y. (3)	x'. (4)	y'. (5)	x''. (6)	$\frac{w\,x}{\Sigma\,w}$. (7)	$\frac{w\,y}{\zeta\,\Sigma\,w}$. (8)	$\frac{w\,x}{\Sigma\,w} \cdot \frac{x'}{\lambda_8}$. (9)	$\frac{w\,y}{\zeta\,\Sigma\,w} \cdot \frac{y'}{v}$. (10)	$\frac{w\,y}{\zeta\,\Sigma\,w} \cdot \frac{x''}{n}$. (11)
1	2.55	0.64	0.67	0.85	0.72	0.63	0.038	0.282	0.003	0.036	0.09
2	2.41	1.94	0.62	2.03	0.67	1.91	0.112	0.246	0.022	0.029	0.26
3	2.28	3.23	0.51	3.29	0.59	3.21	0.173	0.192	0.056	0.020	0.34
4	2.15	4.51	0.36	4.56	0.44	4.47	0.228	0.128	0.108	0.009	0.32
5	2.03	5.79	0.17	5.84	0.34	5.70	0.277	0.056	0.161	0.003	0.17
6	1.93	7.06	0.07	7.10	0.57	7.40	0.321	0.022	0.227	0.002	0.09
7	1.72	8.31	0.37	8.31	0.47	8.39	0.337	0.105	0.280	0.011	0.49
8	1.38	9.57	0.71	9.57	0.77	9.60	0.811	0.161	0.297	0.021	0.85
9	1.19	10.81	1.10	10.81	1.14	10.83	0.303	0.216	0.327	0.043	1.29
10	1.03	12.03	1.53	12.03	1.54	12.04	0.292	0.260	0.351	0.070	1.74
11	2.53	12.88	0.35	13.04	25.54	12.88	0.768	0.146	1.001	0.659	1.04
....	21.20 2	......	......	......	......	......	......	......	2.828 2	0.903 2	......
$\Sigma\,w =$	42.40	......	......	......	......	......	......	......	$v = 5.656$ in.	$n = 1.806$ in.	......

TABLE 17.—COMPUTATIONS FOR MOMENTS IN ARCH A_2 (CENTER SPAN OF A FIVE-SPAN SYSTEM), WITH TWO YIELDING FOUNDATIONS.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.097	− 2.90	− 2.46	−0.281	−0.238	− 2.10	− 1.78	−0.203	−0.172	−0.48	−0.05	− 0.041	−0.094	−2.75	−3.08	−0.266	−0.298
2	0.283	− 3.00	− 2.50	−0.849	−0.707	− 2.12	− 1.79	−0.599	−0.506	−0.33	−0.08	−0.093	−0.022	−2.95	−3.28	−0.834	−0.928
3	0.508	− 3.15	− 2.65	−1.600	−1.346	− 2.14	− 1.80	−1.087	−0.914	−0.20	−0.18	−0.101	−0.091	−3.17	−3.50	−1.610	−1.778
4	0.690	− 3.80	− 3.30	−2.622	−2.277	− 2.17	− 1.81	−1.497	−1.248	−0.25	−0.62	−0.172	−0.427	−4.30	−4.63	−2.967	−3.194
5	0.743	− 4.45	− 3.95	−3.306	−2.934	− 2.20	− 1.82	−1.634	−1.352	−0.65	−1.03	−0.489	−0.765	−5.25	−5.58	−3.900	−4.145
6	0.761	− 5.10	− 4.60	−3.881	−3.500	− 2.25	− 1.83	−1.712	−1.392	−1.28	−1.65	−0.974	−1.255	−6.60	−6.93	−5.022	−5.273
7	0.743	− 5.90	− 5.40	−4.383	−4.012	− 2.05	− 1.65	−1.523	−1.225	−2.38	−2.78	−1.768	−2.065	....	....	....	....
8	0.690	− 6.75	− 6.20	−4.657	−4.278	− 1.55	− 1.12	−1.069	−0.772	−4.00	−4.40	−2.760	−3.036	....	....	....	....
....	0.566	....	....	....	....	....	....	....	....	....	−8.40	....	−4.754	....	....	....	....
....	0.563	....	....	....	....	....	....	....	....	−8.05	....	−4.532	....	....	....	....	....
9	0.508	− 9.65	− 9.05	−4.902	−4.597	− 0.40	− 0.90	−0.203	−0.457	−7.75	−8.07	−3.937	−4.099	....	....	....	....
10	0.283	−16.00	−15.05	−4.528	−4.259	− 7.40	− 8.10	−2.094	−2.292	−7.20	−7.50	−2.037	−2.122	....	....	....	....
....	0.234	....	....	....	....	....	−12.15	....	−2.843	....	....	....	....	....	....	....	....
....	0.230	....	....	....	....	−11.80	....	−2.714	....	....	....	....	....	....	....	....	....
11	0.097	−28.60	−26.65	−2.774	−2.585	−11.70	−12.00	−1.134	−1.164	−7.04	−7.38	−0.682	−0.715	....	....	....	....

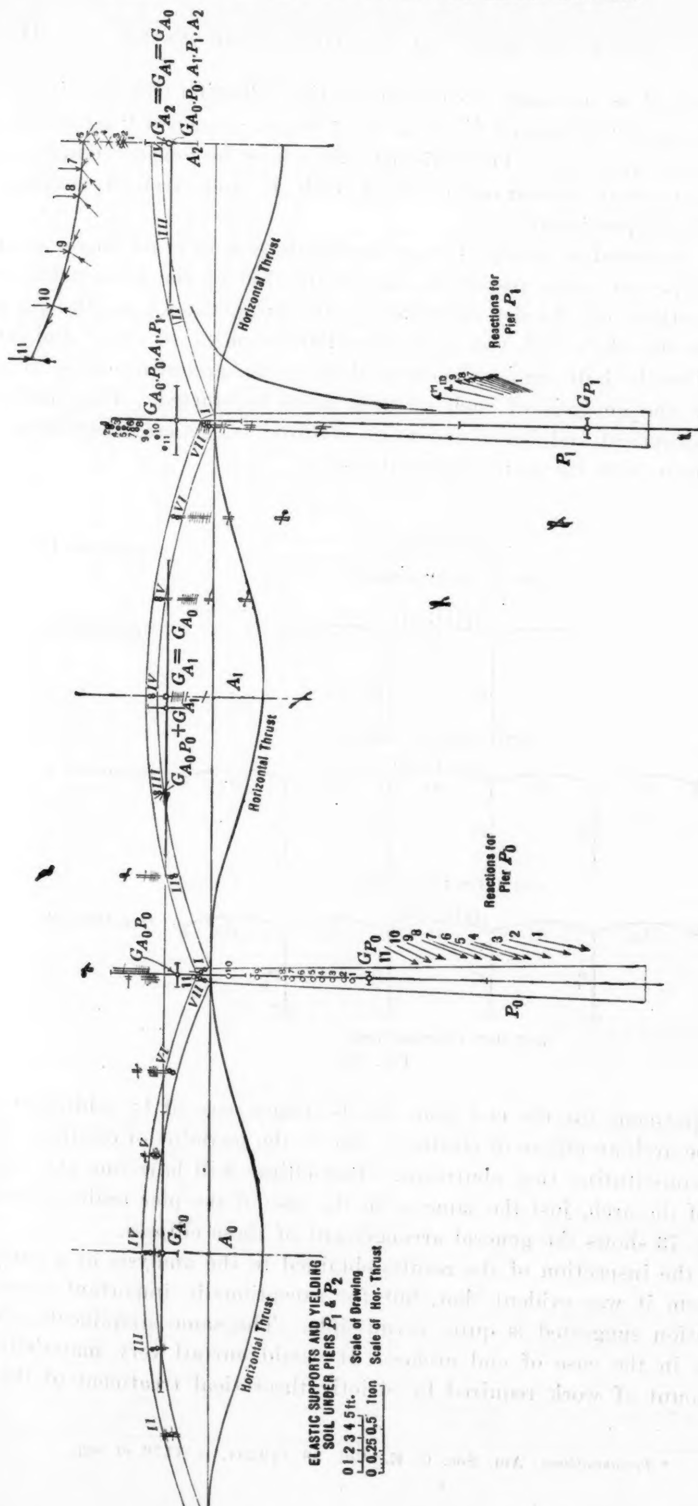

ELASTIC SUPPORTS AND YIELDING SOIL UNDER PIERS P1 & P2
0 1 2 3 4 5 ft. Scale of Drawing
0 0.25 0.5 1 ton Scale of Hor. Thrust
Horizontal Thrust
Horizontal Thrust
Horizontal Thrust
Reactions for Pier P0
Reactions for Pier P1
G_{A_0}
A_0
$G_{A_0} P_0$
P_0
G_{P_0}
$G_{A_0 P_0}+G_{A_1}$
$G_{A_1} = G_{A_0}$
A_1
$G_{A_0 \cdot P_0 \cdot A_1 \cdot P_1}$
P_1
G_{P_1}
$G_{A_2} = G_{A_1} = G_{A_0}$
A_2
$G_{A_0 \cdot P_0 \cdot A_1 \cdot P_1 \cdot A_2}$

FIG. 68.

for instance, it is necessary to determine the following ellipses: G_{A_0}; G_{P_0}; $G_{A_0 P_0}$; $G_{A_0 P_0 A_1}$; $G_{A_0 P_0 A_1 P_1}$; $G_{A_0 P_0 A_1 P_1 A_2}$; $G_{A_0 P_0 A_1 P_1 A_2 P_2}$; $G_{A_0 P_0 A_1 P_1 A_2 P_2 A_3}$; and $G_{A_0 P_0 A_1 P_1 A_2 P_2 A_3 P_3}$. Then this last ellipse must be considered in its correct position, as an ideal voussoir on the left of Arch A_4, and Arch A_4 analyzed as has been shown previously.

It is to be noted, however, that as the elastic system is no longer symmetrical, in this case, with respect to the center line of the geometrical arch, all constructions for the determination of the polygons, p_1, p_3, and p_5, must be done for the whole arch (see Appendix, Paragraph 27 *et. seq.*)* and cannot be limited to the half arch as has been done in the preceding cases. Fig. 73 shows how the analysis of such an arch is to be started. This method is strictly theoretical and hence the work involved is, generally speaking, not commensurate with the usual practical results.

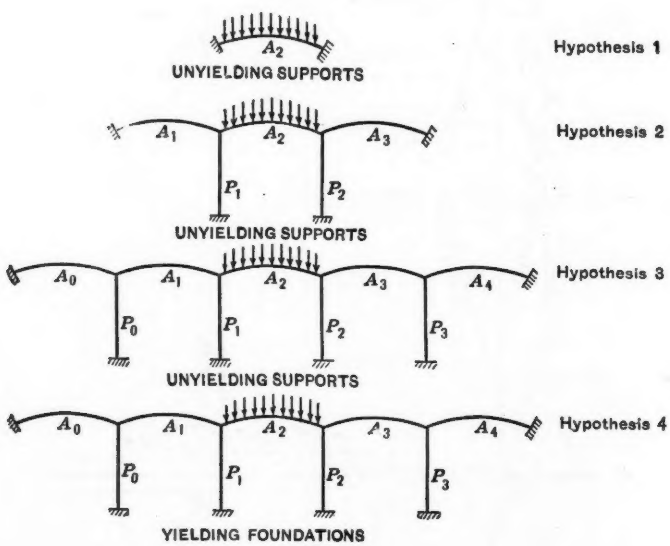


FIG. 69.

The abutment for the end span can be taken care of by adding on that side of the arch an ellipse of elasticity, due to the modulus of elasticity of the material constituting that abutment. This ellipse will have one axis only at the end of the arch, just the same as in the case of the pier built on yielding soil. Fig. 73 shows the general arrangement of these ellipses.

From the inspection of the results obtained in the analysis of a multiple-arch system it was evident that, but for exceptionally important cases, the simplification suggested is quite permissible. The same simplification holds good also in the case of end arches. It would curtail very materially the great amount of work required by strictly theoretical treatment of the end arches.

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1176 *et seq.*

TABLE 18.—COMPUTATIONS FOR MOMENTS IN ARCH A_1 OF FIVE-SPAN SYSTEM WHEN PIERS P_1 AND P_2 REST ON YIELDING FOUNDATIONS.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.083	−4.80	−5.22	−0.398	−0.423	−1.27	−1.62	−0.105	−0.134	−0.87	−0.50	−0.072	−0.041	−2.20	−1.87	−0.182	−0.155
2	0.230	−4.90	−5.33	−1.127	−1.225	−1.35	−1.68	−0.310	−0.386	−0.86	−0.49	−0.197	−0.112	−2.25	−1.92	−0.517	−0.441
3	0.414	−5.08	−5.50	−2.103	−2.277	−1.42	−1.80	−0.587	−0.745	−0.85	−0.48	−0.351	−0.198	−2.32	−1.99	−0.960	−0.823
4	0.553	−5.20	−5.62	−2.875	−3.107	−1.48	−1.85	−0.818	−1.023	−0.84	−0.47	−0.464	−0.259	−2.39	−2.06	−1.321	−1.189
5	0.576	−5.32	−5.73	−3.064	−3.300	−1.55	−1.90	−0.892	−1.094	−0.80	−0.46	−0.478	−0.264	−2.47	−2.14	−1.422	−1.232
6	0.578	−5.55	−5.98	−3.207	−3.456	−1.68	−2.02	−0.971	−1.167	−0.82	−0.45	−0.473	−0.260	−2.58	−2.25	−1.491	−1.300
7	0.551	−5.80	−6.23	−3.195	−3.432	−1.80	−2.13	−0.991	−1.173	−0.81	−0.44	−0.446	−0.242	−2.70	−2.37	−1.487	−1.250
8	0.502	−6.00	−6.40	−3.012	−3.212	−1.90	−2.23	−0.953	−1.119	−0.80	−0.43	−0.401	−0.215	−2.80	−2.47	−1.405	−1.239
9	0.840	−7.23	−7.62	−2.458	−2.590	−2.50	−2.83	−0.850	−0.962	−0.78	−0.40	−0.265	−0.136	−3.38	−3.05	−1.149	−1.037
10	0.146	−10.50	−10.85	−1.535	−1.584	−4.10	−4.40	−0.598	−0.642	−0.65	−0.32	−0.094	−0.046	−5.05	−4.72	−0.737	−0.689
11	0.027	−26.80	−27.00	−0.723	−0.729	−11.90	−12.10	−0.321	−0.326	−0.60	−0.28	−0.016	−0.007	−13.79	−13.46	−0.372	−0.363

Position of load.	H.	Section V.				Section VI.				Section VII.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	−0.083	−2.35	−2.00	−0.195	−0.166	−1.55	−1.11	−0.128	−0.092	−0.50	−0.96	−0.041	−0.079
2	−0.230	−2.47	−2.10	−0.568	−0.483	−1.70	−1.25	−0.391	−0.287	−0.38	−0.80	−0.087	−0.184
3	−0.414	−2.63	−2.27	−1.088	−0.939	−1.95	−1.53	−0.807	−0.633	0	−0.45	0	−0.186
4	−0.553	−2.75	−2.40	−1.520	−1.327	−2.11	−1.68	−1.166	−0.929	−0.26	−0.20	−0.143	−0.110
5	−0.576	−2.90	−2.56	−1.670	−1.474	−2.32	−1.90	−1.336	−1.094	−0.50	−0.08	−0.288	−0.046
6	−0.578	−3.12	−2.77	−1.803	−1.601	−2.65	−2.23	−1.531	−1.288	−0.98	−0.50	−0.566	−0.289
7	−0.551	−3.43	−3.07	−1.889	−1.691	−3.08	−2.65	−1.697	−1.460	−1.55	−1.05	−0.854	−0.578
8	−0.502	−3.62	−3.24	−1.817	−1.626	−3.30	−2.88	−1.656	−1.445	−1.85	−1.38	−0.928	−0.692
9	−0.340	−4.80	−4.40	−1.632	−1.496	−5.07	−4.63	−1.723	−1.574	−4.23	−3.78	−1.488	−1.285
10	−0.146	−8.22	−7.88	−1.200	−1.150	−10.00	−9.55	−1.460	−1.394	−10.80	−10.30	−1.576	−1.503
11	−0.027	−26.00	−25.60	−0.706	−0.691	−35.83	−34.80	−0.953	−0.939	−44.68	−43.95	−1.206	−1.186

TABLE 19.—COMPUTATIONS FOR MOMENTS IN ARCH A_0 OF FIVE-SPAN SYSTEM WHEN PIERS P_1 AND P_2 REST ON YIELDING FOUNDATIONS.

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.087	−2.96	−3.40	−0.257	−0.295	−0.40	−0.80	−0.084	−0.069	−0.88	−0.54	−0.076	−0.046	−1.30	−0.97	−0.113	−0.084
2	0.238	−2.90	−3.35	−0.690	−0.797	−0.37	−0.77	−0.088	−0.183	−0.89	−0.54	−0.211	−0.128	−1.28	−0.95	−0.304	−0.226
3	0.435	−2.84	−3.30	−1.235	−1.435	−0.34	−0.74	−0.147	−0.321	−0.90	−0.54	−0.391	−0.234	−1.26	−0.93	−0.548	−0.404
4	0.584	−2.78	−3.25	−1.623	−1.898	−0.31	−0.71	−0.181	−0.414	−0.91	−0.54	−0.531	−0.315	−1.24	−0.91	−0.724	−0.531
5	0.608	−2.72	−3.20	−1.653	−1.945	−0.28	−0.68	−0.170	−0.413	−0.92	−0.55	−0.559	−0.334	−1.22	−0.89	−0.741	−0.541
6	0.617	−2.66	−3.15	−1.641	−1.943	−0.25	−0.65	−0.154	−0.401	−0.93	−0.56	−0.604	−0.345	−1.20	−0.87	−0.740	−0.536
7	0.593	−2.60	−3.10	−1.541	−1.838	−0.22	−0.62	−0.130	−0.367	−0.94	−0.57	−0.557	−0.338	−1.18	−0.85	−0.699	−0.504
8	0.542	−2.54	−3.05	−1.376	−1.653	−0.18	−0.58	−0.097	−0.314	−0.95	−0.58	−0.514	−0.314	−1.16	−0.83	−0.628	−0.449
9	0.380	−2.48	−3.00	−0.942	−1.140	−0.13	−0.54	−0.049	−0.205	−0.96	−0.59	−0.364	−0.224	−1.14	−0.81	−0.433	−0.307
10	0.178	−2.40	−2.83	−0.427	−0.503	−0.08	−0.50	−0.014	−0.089	−0.97	−0.60	−0.172	−0.106	−1.12	−0.79	−0.199	−0.140
11	0.046	−1.50	−1.95	−0.069	−0.089	−0.40	−0.02	−0.018	−0.009	−1.10	−0.74	−0.050	−0.034	−0.79	−0.46	−0.036	−0.021

Position of load.	H.	Section V.				Section VI.				Section VII.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.087	−0.50	−0.15	−0.043	−0.013	−1.15	−1.54	−0.100	−0.133	−4.12	−4.55	−0.358	−0.421
2	0.238	−0.46	−0.11	−0.109	−0.026	−1.22	−1.61	−0.290	−0.383	−4.22	−4.65	−1.004	−1.106
3	0.435	−0.42	−0.07	−0.182	−0.030	−1.29	−1.68	−0.561	−0.730	−4.32	−4.75	−1.879	−2.066
4	0.584	−0.38	−0.03	−0.221	−0.017	−1.36	−1.75	−0.794	−1.022	−4.42	−4.85	−2.581	−2.832
5	0.608	−0.33	−0.02	−0.200	−0.012	−1.43	−1.82	−0.869	−1.106	−4.52	−4.95	−2.748	−3.009
6	0.617	−0.28	−0.07	−0.172	−0.043	−1.51	−1.89	−0.931	−1.166	−4.62	−5.05	−2.850	−3.115
7	0.593	−0.23	−0.12	−0.136	−0.071	−1.59	−1.96	−0.942	−1.162	−4.73	−5.16	−2.804	−3.059
8	0.542	−0.18	−0.17	−0.097	−0.092	−1.67	−2.04	−0.905	−1.105	−4.84	−5.27	−2.623	−2.856
9	0.380	−0.12	−0.23	−0.045	−0.087	−1.76	−2.12	−0.668	−0.805	−4.96	−5.38	−1.884	−2.044
10	0.178	−0.05	−0.30	−0.008	−0.053	−1.85	−2.22	−0.329	−0.398	−5.08	−5.50	−0.904	−0.979
11	0.046	−0.70	−1.05	−0.032	−0.048	−2.95	−3.30	−0.135	−0.151	−6.60	−7.00	−0.303	−0.322

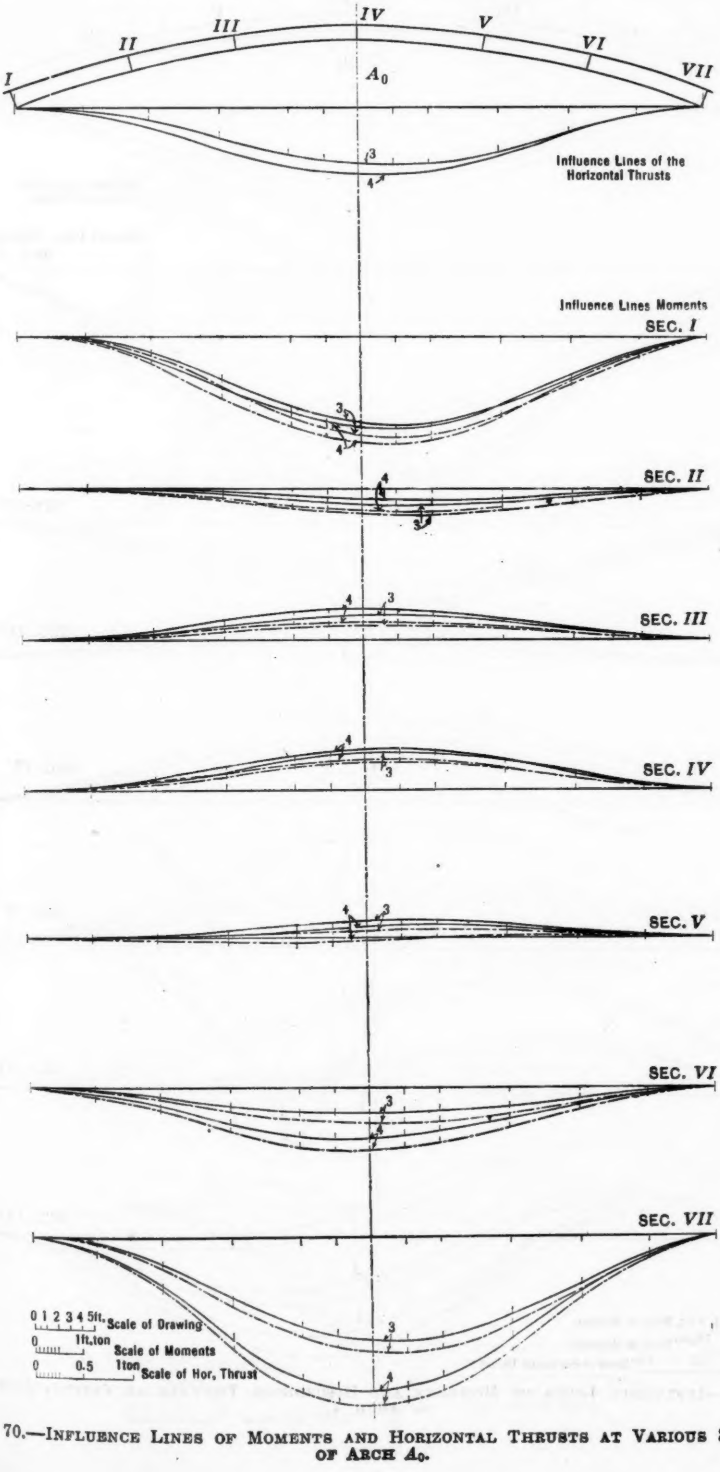


FIG. 70.—INFLUENCE LINES OF MOMENTS AND HORIZONTAL THRUSTS AT VARIOUS SECTIONS OF ARCH A_0.

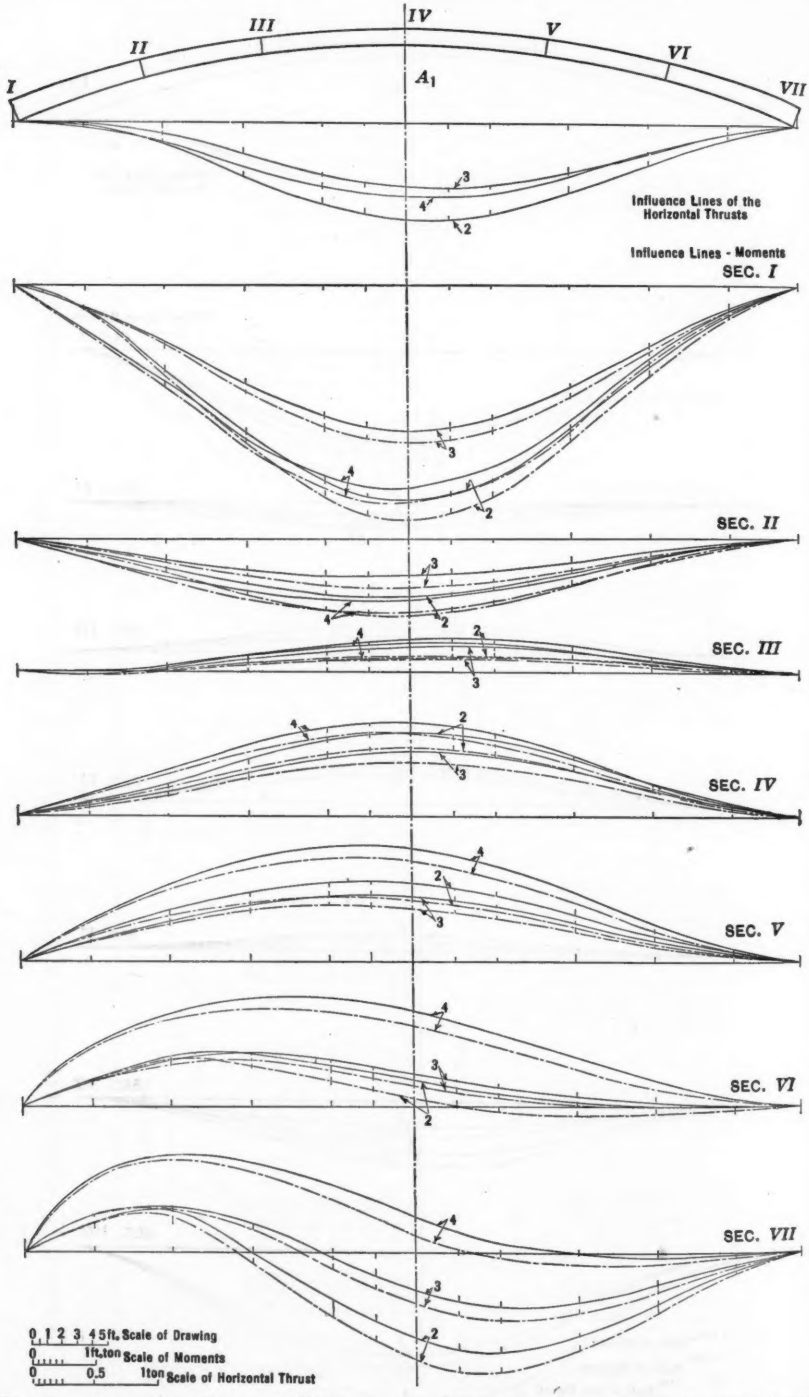
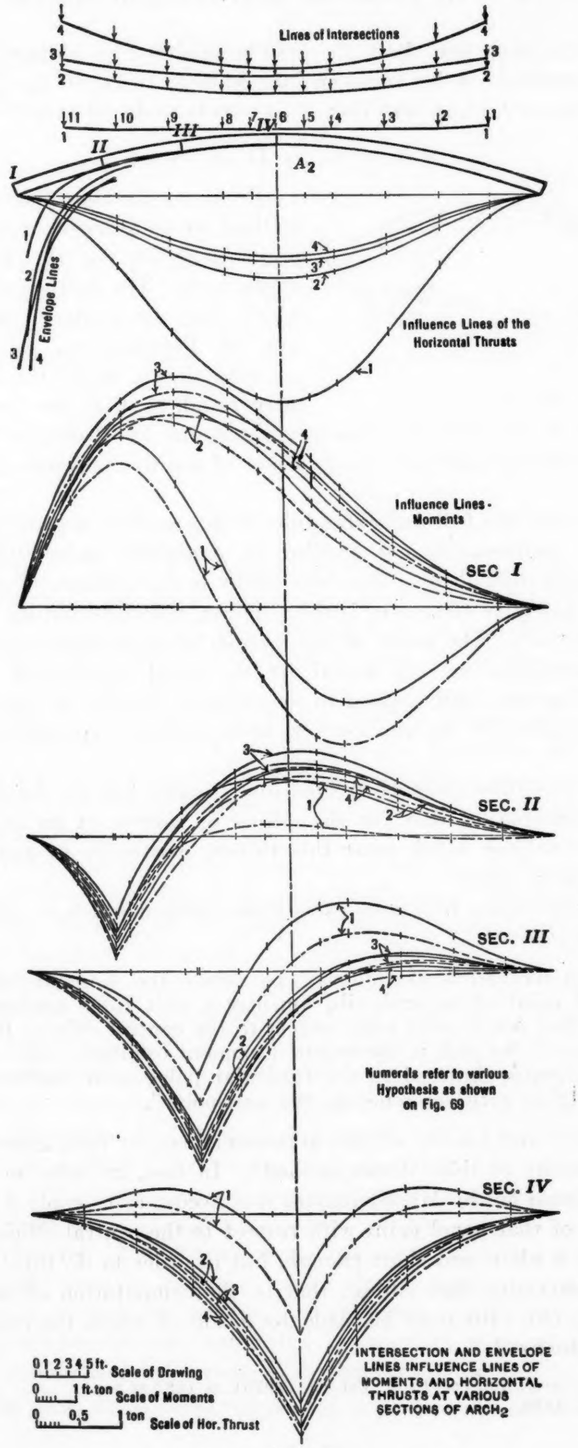


FIG. 71.—INFLUENCE LINES OF MOMENTS AND HORIZONTAL THRUSTS AT VARIOUS SECTIONS OF ARCH A_1.



Lines of Intersections
Envelope Lines
Influence Lines of the Horizontal Thrusts
Influence Lines - Moments
SEC I
SEC. II
SEC. III
Numerals refer to various Hypothesis as shown on Fig. 69
SEC. IV
INTERSECTION AND ENVELOPE LINES INFLUENCE LINES OF MOMENTS AND HORIZONTAL THRUSTS AT VARIOUS SECTIONS OF ARCH 2
Scale of Drawing
Scale of Moments
Scale of Hor. Thrust

FIG. 72.

In this case, therefore, Arch A_4 could be analyzed by adding to the various ellipses of elasticity of the voussoirs, the ellipse, $G_{A_3P_3}$ ($G_{A_3P_3} = G_{A_0P_0}$), of the joint between Arch A_3 and Pier P_3, which is readily determined.

Comments on Discussions*

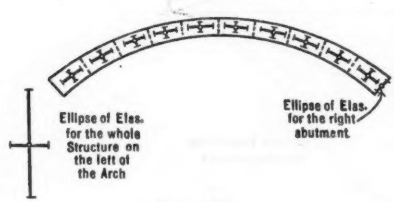


Fig. 73.

Professor Rathbun has discussed this method in comparison with others followed generally in the analysis of a single arch. The further simplifications which may be made in the construction of Polygons p_1, p_2, p_3, p_4, and p_5, here shown, make this method still more rapid, so that the saving of time of one-third to one-fifth, as already pointed out by Professor Rathbun, in the case of a full geometrical construction of the five polygons, becomes still greater.

It is true that the full understanding of this method requires some knowledge of the mathematics not studied in American universities, but this knowledge is not difficult to obtain from books on this subject. Probably, of all geometries, Projective Geometry is the simplest, notwithstanding its "discouraging expressions". The study of this branch of mathematics is not an idle one; such knowledge is very useful for the rapid solution of problems of Descriptive Geometry and, applied to complicated theories, as for instance the one in point, makes its explanation very clear, and its application simple and rapid.

However, this theory can be treated analytically, but its demonstration is long and wearisome, as that for the ellipse of inertia of an area. Several works on this subject which treat this theory, mostly in an analytical way, have already been noted.

Mr. Steinman refers to a more expeditious method which he calls a "direct method". He states:†

"The direct method is as follows: To obtain the moment influence line for any panel point of an arch rib, consider a unit force applied along the anti-polar of that panel point with respect to the central ellipse; the resulting deflection curve of the arch is the required moment influence line. This deflection curve is simply obtained as the funicular polygon of static moments of the elastic weights with reference to the anti-polar".

The neatness and brevity of this statement may, at first, convey a wrong idea of the brevity of this "direct method". In fact, in order to follow this method, as pointed out by Mr. Steinman, it is necessary to apply a force along the anti-polar of that panel point with respect to the central ellipse; verbally, this operation is plain and short enough, but in order to do this, it is necessary first to determine that ellipse; that is, the computation of at least Columns (7), (8), (9), (10) must be made, by means of which the central ellipse is fully individualized.

* See *Transactions*, Am. Soc. C. E., Vol. 88, (1925), p. 1183 *et seq.*

† *Loc. cit.*, p. 1186.

The writer, in showing how the position of the reactions can be determined, has preferred a graphical construction which gives, of course, the same result as when the position of the reaction is determined by the method suggested by Mr. Steinman, with the only difference that the construction preferred by the writer is quicker and simpler. The paper also noted that there are several methods for determining this position.

That the method preferred by the writer is far shorter and simpler than that suggested by Mr. Steinman can be seen by the fact that for each position of the reaction (considered as anti-polar) it is necessary first to determine the corresponding pole (center of instantaneous rotation of the free end of the arch under that assumption of loading), the co-ordinates of which are given by:

$$x = \lambda_3 \frac{\eta_1}{\eta_0}$$

$$y = \xi\, n \frac{\eta_2}{\eta_0}$$

while the method followed by the writer involves one computation for each position. Likewise of the graphical constructions, that in the method preferred by the writer is the simpler. Thus it may be seen that the method suggested by Mr. Steinman is nothing but a part of the standard method, and, therefore, it is incomplete.

The enunciation of the principle of the relation between the line of influence and the deflection diagram, as given by Mr. Steinman, does not seem exact. He states, "every influence line is a deflection diagram", which can be taken to mean that the load diagram, for instance, is a deflection diagram in itself, and this would not be correct.

There are two graphical methods to demonstrate the relation between the load diagram and the corresponding moment diagram. Saviotti* obtained, for the first time, the diagram of moments from the diagram of loads by the method called "graphical integration". He demonstrated how it is possible to obtain the diagrams of shear, of moments, and of tangents, and the deformed elastic axes of a beam, by successive graphical integrations of the diagram of loads. The other method is called the "funicular polygon method" by means of which it is possible to shorten the construction involved in the first method so as to obtain the diagram of moments directly from the diagram of loads.

The writer must confess that, in presenting his paper, he took for granted that it was only necessary for him to show the extent of variation in the moments of an arch in the two hypotheses assumed in the paper, namely, an arch on unyielding supports and an arch on yielding supports; and that the yielding part could be considered as constituted by an arch and a pier on each side. He thought that the extension of the elastic supports to more than an arch and a pier was almost useless, for engineers could forecast that the results would not be changed materially, if they had followed mentally the process of distribution of the forces on the lateral arches.

* "Sul metodi grafici d'integrazione," C. Saviotti, *Il Giornale del Genio Civile*, 1882.

In reading some discussions, including that of Mr. Steinman, he has realized that he may have taken too much for granted, and he willingly has supplied what he thinks was wanted. The writer, however, did not overlook "the possibility (which may upset his conclusions) of the two outer spans being loaded simultaneously". From Plate III,* or Tables 7† and 8,‡ it is seen that the maximum moment at Section I for Arch A_2 is verified when Arches A_1 and A_3 are loaded and Arch A_2 is unloaded, and, therefore, as stated in the paper, the maximum moment at Section I for Arch A_2 would be given by twice the total moment obtained from the diagram for Section I of Arch A_2.

Theoretically speaking, this is true, but from a practical point of view it may be disregarded. An assumption of loading must always correspond to practical exigencies and possibilities of the various combinations of live load crossing a bridge. Now, except for street cars, which in a very abnormal case could fill up both tracks on the bridge on each side of the central span leaving this last one unloaded, the remaining live load, pedestrians, trucks, etc., cannot, for all practical purposes, be distributed according to the assumption suggested.

A dense crowd passing over a bridge is divided approximately into two parts, one-half going and one-half coming, therefore, the most prejudicial hypothesis of loading, in this case, must be that the two side arches are loaded each with one-half the live load. Even then this rather abnormal condition of loading would be of a very short duration, for the crowds keep moving. Hypotheses of loading are circumstantial.

A scientific discussion is very apt to bring forth different points of view; but, when these are candid, as Mr. Godfrey's are, the conflict of opinions is most useful to the reader, who then is able to weigh the arguments on both sides. It is true that there are many causes for uncertainty regarding the relation between the assumptions made in the analysis of an arch and the actual conditions, but it must be said that they have been strongly exaggerated by Mr. Godfrey in his discussion.

The limits of variation between the behavior of the characteristics of the various kinds of concrete, in general, are not so far apart, and the attention of Mr. Godfrey is called to the fact that an engineer's assumptions always are, or should be, based on the most unfavorable case. In addition, the writer reduced such assumptions by approximately one-quarter. This seems conservative enough.

According to Mr. Godfrey, Professor G. Lanza has found that "two identical beams showed results almost 100% apart", but Professor Lanza does not state whether this enormous difference was verified below or above the usual limits for the resistance of concrete. Concrete made in laboratories is not, in general, the same as that obtained in fairly well organized construction, where modern mechanical means have reduced the influence of the human factor to a minimum, and where a satisfactory uniformity for the concrete is an accomplished fact.

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1145.
† *Loc. cit.*, p. 1155.
‡ *Loc. cit.*, p. 1157.

The hypothesis added by Mr. Godfrey concerning the annealed glass arch, gives a fair idea of his frame of mind on this subject—he is striving for perfection. Now, why should arch design be perfect, while no other structural design is perfect? Every engineer knows that he cannot analyze a structure perfectly; all methods of analysis are mathematically correct, under the assumptions, but their application to this or that structure is always approximate (safe within certain limits). This is due to the fact that some physical phenomena in the structure, thus far, have been found to be of such a difficult and complex nature as to make them uncontrollable by analysis.

If sophism is allowed to invade engineering practice, the whole profession will be unjustly discouraged. The writer is unable to accept the contention that a reinforced concrete arch cannot be considered as an elastic arch. After the results obtained from the tests of the Austrian Committee,* which have demonstrated beyond a doubt that arches made either with a natural or artificial stone fall well within the limits of elasticity, it does not seem necessary to recall any other argument to oppose this contention of Mr. Godfrey.

Concerning the abutments, it is not necessary to add "wasteful mass" to make them practically unyielding. It is for the designer to investigate whether it is more economical to add concrete to the abutment to render it unyielding, or to analyze an arch on the assumption that the abutment yields.

Mr. Godfrey is right when he states that the fixedness of the supports of an arch is seldom verified; but the writer thinks that for practical purposes many abutments may be assumed as unyielding. As previously shown, there is a way of taking care of a rotation of the abutment, when necessary.

Mr. Godfrey advocates the analysis of an arch on the assumption that it be made up of voussoirs, instead of being monolithic as it is. This procedure would put engineering back more than half a century, and would eliminate only a few of the misgivings mentioned by Mr. Godfrey, with the additional inconvenience of making arches unjustifiably heavy. If the quantity of materials used in abutments is "wasteful", certainly this practice would be extravagant.

There are also scientific reasons for not following this theory. By analyzing an arch system under the old theory of the single arch, it would be difficult to investigate the effects on an arch produced by the loading of another arch of the same system. Mr. Godfrey cannot deny that an arch, under the action of loading, deforms. If so, the arch is an elastic solid and, as such, its analysis may be carried out by the theory of elasticity.

Mr. Harder's argument seems to be that since certain engineers use a given method, all other engineers should follow the same procedure. He states that, since the ellipse of elasticity is not studied in certain schools, it should not be used by engineers; he also adds: "However, the pendulum may swing in the other direction, the ellipse of elasticity may be dusted off, and again the polars will rotate about the anti-poles".

* Bericht des Gewölbe Ausschusses, *Zeitschrift* des Osterreichischer Ingenieure und Architekten Vereines, Wien, 1895.

The matter of a scientific principle has no connection with fashion. Whether or not the principle is a good one is of most moment, rather than whether or not certain schools are using it. In the writer's opinion, the ellipse of elasticity will always remain at its own scientific height, helping engineers in the rapid solution of complicated problems for which the application of the analytical methods would be either unpractical or extremely difficult.

Mr. Harder criticizes this theory because it does not take shear into consideration, citing the opinion* of George F. Swain, Past-President, Am. Soc. C. E. The writer recommends that Mr. Harder be not misled by the style which Professor Swain chose to adopt in those remarks but that he read Paragraphs 21,† 21 (*a*),‡ and 21 (*b*),§ of the Appendix, which cover this matter completely. From these demonstrations it is hard to understand how such an objection can have any weight whatsoever.

There is no need of any classification of structures "in which the influence of the web members on the deflection of the structure is negligible". This method is quite general, and depends on the skill of the designer to take or omit the consideration of shear in his particular problem. Science cannot be reduced to recipes and prescriptions. That there may be cases in which the deflection due to web members may reach 41%, as found by Ritter, nobody will deny; but these are very rare, and any designer, to be worthy of the name, must be able to distinguish.

Mr. Harder has some criticism to make on this method, "because of inaccuracies and awkwardness of some of the construction (for example, the small force polygon with a pole at P_5‖). Any error in this small force polygon may mean a large error in the inflence line for the horizontal thrust, p_5." It should be noted that the force polygon, P_5, may be multiplied by a constant to make it as large as the force polygon, P_1, or larger. In connection with Table 12 the writer has shown a simplification whereby this polygon need not be drawn.

Concerning the use of graphical methods of analysis, there is some ill-founded mistrust among those who believe that analytical methods are far more exact, in any case. There are causes of error in analytical, as there are in graphical, computations. The mathematical concessions that a designer is compelled to make in his analytical design, in order to render it practical, are causes of errors. Although his arithmetical computations are correct, they represent results only approximate, on account of simplifications adopted at the beginning of his analysis, as well as during the working out of his formulas.

* "On a New Principle in the Theory of Structures," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 702 *et seq.*

† *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1172.

‡ *Loc. cit.*, p. 1173.

§ *Loc. cit.*, p. 1174.

‖ In Mr. Harder's discussion (*Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1194), this is incorrectly given as p_5.

Graphical methods, if well adapted to the problem in point, do not need so many approximations, and they are quick, plain, and uniform; and errors depending on bad draftsmanship can be easily detected and corrected. That the graphics are sufficiently exact is shown by the fact that the results found graphically by the writer, have been checked by laboratory results found by Professor Beggs, with a close approximation that analytical methods of design would find rather difficult to attain.

The writer never had the idea of presenting a complete application of this theory to a multiple-arch system. What he had in mind was whether, in dealing with a multiple-arch system, the consideration of only three arches of the system was permissible. It is true as Mr. Harder notes that in each multiple-arch system there are two end arches and that the stresses in these arches will be different from those of the central arch. The writer assumed that every engineer would take this for granted.

Professor Cross' discussion is marked by many incorrect statements, and faulty applications of mathematical principles. The writer will point out only a few errors, each of which is vital to some conclusion which Professor Cross has derived from it.

The name of this theory was given to it at a time when the writer was a mere infant. The first scientist (Culman) who announced this theory called it "the ellipse of elasticity" and after him scores of other eminent men called it by the same name, so that the writer disclaims all the responsibility for this name, although he finds it quite justifiable.

Professor Cross thinks that Professor Guidi's assumption, as stated in the paper, does not necessarily involve fixed-endedness for the flanking arches. To this there is but one reply, and this is in Professor Guidi's own words.*

The contention that the theory of the ellipse of elasticity does not take the shearing stress into consideration, and that, therefore, it is not suitable for the calculation of web members of an elastic system, has already been answered. It is interesting to note that other discussors including Professor Mylrea, having read the same paper, have understood clearly how this theory takes into consideration the web members, if this is desired.

As previously noted Paragraph 21 of the Appendix covers this point and shows clearly how the shearing stress is taken care of in the case of a solid web. Furthermore, from Paragraphs 21 (*a*) and 21 (*b*) of the Appendix, Professor Cross may see how to take care of the web members of a latticed arch.

Therefore, when he says "This is not clear; indeed the discussion of trusses in this paper seems entirely irrelevant", apparently he has failed to see that the elastic weights are to be applied to the center of gravity of each voussoir, in the case of solid ribs, and of latticed ribs with parallel flanges; and to be applied instead at the poles, in the case when the flanges of a latticed member are not parallel as shown in Fig. 16 (*d*)†. If the designer intends to take shearing stress into consideration, it is only neces-

* "Teoria dei Ponti," Camillo Guidi, Fourth Edition, p. 473.

† *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1174.

sary to compute the major axes of the elemental ellipses of elasticity of a system by the formula,

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{E}{S} \times \rho^2}$$

instead of by

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

in the case of solid rib; by the formula

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{A \times s^3}{2\,A'\,.f}}$$

instead of

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

in the case of a type of latticed girder with parallel flanges; by the formula,

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{A\ .\ s^3}{2\,A'\ .\ f} + \frac{A\ .\ h^3}{2\,A''\ .\ f}}$$

instead of

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

in the case of another type of latticed girder with parallel flanges; and, finally, in the case of girders with non-parallel flanges, the designer should follow what is said in Paragraph 21 (*b*). This may "seem irrelevant" to Professor Cross, but most engineers would not agree with him.

In most engineering problems, certainly, it is not necessary to take into consideration the deformation of web members. This is merely a general statement that holds good except in those rare cases in which, either on account of the importance of the elastic system, or on account of the nature of the work that this system is assumed to perform, the designer recognizes the necessity of considering the web members. These are not every day cases, however, and it is the judicious discrimination of problems that determines a designer's standing. Everybody who is familiar with designing knows that there are elastic systems or members of elastic systems that undergo large shearing stresses; but these are exceptions and not a rule.

Apart from Professor Cross' opening remarks, already treated, his discussion, on the whole, can be considered as offering a set of formulas which are "perfectly general" in their application and which, in the opinion of Professor Cross, could be used in place of the method shown by the writer.

On the contrary, it may be demonstrated that:

First.—Assuming the correctness of the formulas, their actual application would require far more labor than the writer's method.

Second.—The formulas themselves are wrong.

Professor Cross starts by assuming that I_a, J_a, and Z_a are already known to the designer by the time he has finished the first step of his work, that is, the analysis of the arch and pier separately.

From Fig. 20 * (whether it be a latticed arch or a solid rib arch is immaterial to this discussion) it is seen that after such an analysis the designer knows the value of I_a about the vertical diameter, y, of the ellipse; that of the moment of inertia about x conjugate with y; and $Z_{yx} = 0$. Now x is never horizontal, except in the case of a symmetrical arch, and Professor Cross' research is for a "perfectly general" application of his formulas. Therefore, the designer will have to compute J_a about the horizontal, x', Fig. 74, which is by no means an easy task. Similar remarks apply in regard to the value of Z_a.

By exactly the same line of reasoning it can be shown that the designer will have to compute I_s, J_s, and Z_p, before using Professor Cross' formulas, since the ellipse, s, of the Joint B in a "perfectly general" case, is determined by finding the position of two conjugate diameters, which (except for a symmetrical structure) are never either horizontal or vertical. Only for the pier would the values required by the formulas be already at hand. Thus, the application of these formulas, assuming their correctness, would entail by far a greater amount of labor than the method shown by the writer.

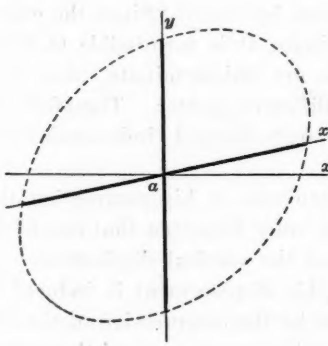


FIG. 74.

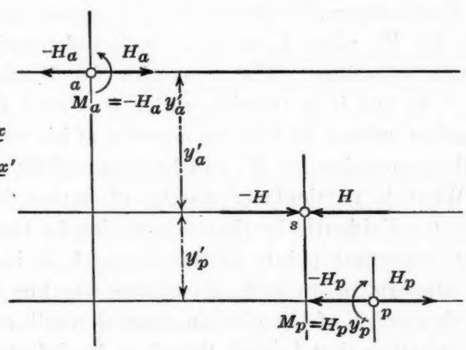


FIG. 75.

Concerning the mathematics involved in deriving these formulas: In Fig. 24,† Professor Cross assumes that a is the elastic center for the arch; that p is the elastic center for the pier; and that s is the elastic center for the system (arch and pier). Point s is not the elastic center for the system, but it is the elastic center for Joint B, between the arch and the pier, which is quite a different thing.

In Fig. 25 (a)‡ Professor Cross shows a certain condition of equilibrium. He applies an horizontal force, $H = 1$, at Point s and two horizontal reactions one at a and one at p with a couple, V, stating that his system is in equi-

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1177.

† *Loc. cit.*, p. 1199.

‡ *Loc. cit.*, p. 1200.

librium. If, as he states, $H_a + H_p = 1$, it is difficult to understand what couple will balance the couple, V. The presence of the couple, V, means a rotation of the system, and there is no rotation; therefore, equilibrium requires a couple, $-V$, which is missing.

Having assumed that $H_a + H_p = 1$, he implicitly states that the action of the force, $H = 1$, at s, has been balanced; it is therefore logical to assume that the couple, V, does not exist, unless there is a claim to perpetual motion.

In Fig. 75, is shown how the equilibrium of the system should be considered. Applying at s one horizontal, force, H, a force, $-H$, would counteract it and the system would be in equilibrium. Now by dividing the force, H, into two parts, inversely proportional to the distances, y_a' and Y_p', they may be imagined as properly transferred, one at a and one at p; in order not to alter the equilibrium, imagine the forces, $-H_a$ and $-H_p$ applied at a and p. Thus, the equilibrium has not been altered.

This transferring and addition of forces, however, has created two couples; one, $M_a = -H_a y_a'$, about a and one, $M_p = H_p y_p'$, about p, which, obviously, are equal and counterbalance each other. The action, therefore, of the force, H, is counteracted by H_a, H_p, M_a, and M_p. Unfortunately, the formulas given by Professor Cross have been derived on the condition of equilibrium as given in his Fig. 25 (a), and, therefore, they cannot be correct.

Furthermore, Professor Cross is wrong when he states: "Since the expression for W_s when I_p or $J_p = 0$, is indeterminate, it is permissible to write" certain equations. The expression for W_s is not indeterminate, when I_p or $J_p = 0$; but it is infinite, which is a very different matter. Therefore, the equation written by him, on account of his wrongly claimed "indetermination" of the expression for W_s can have no weight.

What is particularly worthy of notice, however, is his assumption that $I_p = 0$. Evidently by that assumption he has quite forgotten that one of the most important points of this research is just the vertical displacements of the supports of an arch, no matter whether this displacement is induced by the shortening of the pier, in general small, or by the compression of the soil. By assuming that $I_p = 0$, therefore, he defeats the very purpose of the method as presented by the writer.

Since all these points have been demonstrated to be utterly incorrect, this formula, like the preceding ones, is to be disregarded. By using these formulas, Professor Cross finds data for the pier and fixed-ended arch, which are not very dissimilar from the corresponding data found by the writer. That incorrect methods and formulas should give approximately correct results is simply fortuitous.

Professor Cross then gives a questionable application of the principle of reciprocity to an elastic system. Fig. 27 (a)* shows an arch for which he proposes to find the vertical deflection of Point b (the key of the arch). In order to find this vertical deflection, he applies a unit horizontal force at Point a (elastic center of the arch) and states that the "vertical deflection of

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1204.

$b = i_h \cdot J_a$, where i_h is the influence ordinate at b for the horizontal thrust on the fixed arch." This, to say the least, is staggering.

Every engineer knows that, by the principle of reciprocity, the vertical deflection of b is found by applying a vertical unit force at b, and by computing the horizontal displacement of Point a; this horizontal displacement of a will be the vertical displacement of b when the horizontal force, 1, is applied to a. Now, the horizontal displacement of a, due to a vertical force, 1, acting on b, is given, according to the theory of the ellipse of elasticity, by the force multiplied by the product of inertia of the elastic weights of the arch on the left of b (if it is assumed that the left end of the arch is fixed, and the right end is free, and that the center, a, is connected rigidly with the free end of the arch); that is, by $1 \cdot \lambda_2 \cdot n \cdot \eta_2$. (See Paragraphs 17,* 33,† and 37‡ of the Appendix, and the top of page 1146.§)

It is this value which, by the principle of reciprocity, represents the vertical displacement of b when the horizontal force, 1, acts on a. Clearly it is an altogether different value from the one given by Professor Cross. What he did was to imagine that he had found the value of the horizontal displacement of Point a, under the action of a unit horizontal force, and to imagine that, by so doing, he had also found the vertical displacement of b.

There are two very grave errors in his method. The first is the wrong application of the well-known principle of reciprocity, as shown; the second is that the expression, $i_h \cdot J_a$, as given by Professor Cross, and supposed by him to be the vertical displacement of Point b is nothing but the horizontal displacement of the elastic center, a, under the action of a horizontal force, i_h. (See Paragraphs 25,‖ 26¶ and 41** of the Appendix.) As a matter of fact the horizontal displacement of the elastic center, a, under the action of the horizontal force, 1, is $1 \times J_a$.

Whatever the explanation, the fact remains that his value is perfectly foreign to the case in point. This is not a case of bringing b to its former position, which return could be obtained by applying at a a horizontal force, H (i_h of Professor Cross), and, consequently, causing a horizontal displacement of $a = H \cdot \lambda_2 \cdot v \cdot n$ ($i_h \cdot J_a$ of Professor Cross). This is a case of finding the vertical displacement of b when a horizontal force, 1, acts on a, no matter how much it is going to be displaced. The difference between these two cases is really quite clear, the "obfuscation" being purely in Professor Cross' mind.

There is a final mathematical question treated by Professor Cross which certainly needs explanation, when he states††: "Also, a unit moment at a will produce a vertical deflection at b equal to $i_m\ W_a\ e_y'$, in which, i_m is the influence ordinate at b for the moment at the neutral point of the fixed arch".

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1171.
† *Loc. cit.*, p. 1178.
‡ *Loc. cit.*, p. 1179.
§ *Transactions*, Am. Soc. C. E., Vol. 88 (1925).
‖ *Loc. cit.*, p. 1175.
¶ *Loc. cit.*, p. 1176.
** *Loc. cit.*, p. 1179.
†† *Loc. cit.*, p. 1203.

This is possibly even more questionable than the point just mentioned. It is well-known that by the principle of reciprocity, the vertical displacement of Point b, caused by moment $= 1$, acting on the elastic center, a, is given by the rotation of the center, a, caused by the vertical force, 1, applied at b. Now, applying a vertical force, 1, at the point, b, gives a rotation of the center, a, and this rotation, by the theory of ellipse of elasticity, is given by $1 \; \Sigma \; W \; \eta_0$, in which, η_0 is the ordinate moment of the elastic weights of the arch from the left up to Point b (if it is assumed that the arch is fixed at the left, and free at the right, as previously).

This value represents the vertical displacement of b when a unit moment is applied at a. Thus, it is seen that the correct value here given differs from that of Professor Cross, by a certain distance, e_y,* which however, has been introduced with no apparent justification. Subsequently, Professor Cross finds that the value of this quantity $e_y = 0.10$, that is, that the value of the vertical displacement, b, as found by him is only one-tenth of what it should be.

It is with such formulas that he undertakes the "Study of Crown Thrust".† No other comment is necessary, except to say that evidently Table 9,‡ with all Professor Cross' assumptions, needs a radical revision.

Thus, it seems that Professor Cross, with very little consistency, has treated with contempt the theory of the ellipse of elasticity, while on the other hand he has used and misused it throughout his camouflaged algebraic method; but that he should commit errors in other theories which apparently are accepted by him, seems unpardonable.

Professor Cross states§, finally, "the writer believes that a correct and practically complete analysis of a multiple-arch system is quite feasible," etc. This is merely a statement, and in mathematics a statement has no value if not supported by demonstration. It would have been quite proper for him to have presented a practical illustration which would have been entitled to consideration.

As to the statement‖ in the beginning of his discussion that, in America, "college graduates know little about properties of poles, polars, and anti-poles, and are, perhaps, not the worse for the deficiency", if college graduates know so very little about these properties that they cannot grasp the meaning of principles founded on them, it is scarcely worth while wasting their time in studying the subject at all.

Mr. Hammill, who has used this method for several years, has found it to be more accurate than an analytical one. His conclusion could not be otherwise. Those in favor of exclusively analytical methods apparently forget the approximations they have to make in solving their equations. Furthermore, when a graphical method can be checked, at any moment, by simple slide-rule computations, all criticisms against the method merely show a lack of familiarity in more advanced theories.

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* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 1204. In Fig. 27 (*b*) of Professor Cross' discussion, "c_y" should read "e_y." Text is correct.

† *Loc. cit.*, p. 1203.

‡ *Loc. cit.*, p. 1205.

§ *Loc. cit.*, p. 1206

‖ *Loc. cit.*, p. 1197.

A well-known professor once said, in substance, that graphical methods were unreliable for even the thickness of a line may mean a difference of value. The writer is not sure if this professor had in mind the lines drawn by carpenters in their daily work, or lines drawn by any average designer. In the first case he was right; in the second apparently he was not quite familiar with graphics.

Professor Beggs has supplied this paper with a very interesting complementary work. The writer will not enter this field for it is so clearly explained by its author. His method for testing models is ingenious and his results as regards the triple arch treated by the writer are very gratifying, for they show that the analysis of this system has been correctly applied. These results seem to answer the criticism that graphical methods are inaccurate.

The difference recorded in the diagrams between the theoretical values of the moments and the observed values of the moments on the template, in the writer's opinion, are due to unavoidable imperfections of the small model, and to some slight friction which develops in its deformation during the test.

Mr. Hudson is right when he calls "unusual" the type of structure taken for example by the writer; but, evidently, he has not thought that this structure was purposely taken as an example, for its make-up is such that the difference in results obtained from the various combinations of the arches would be more apparent. There was no intent of preparing plans for a system of arches to be built for every-day use. The idea was to present a method to be applied to a system of arches. In many books the diagrams explaining theories and methods of design are not drawn for actual building, but to bring home the truth in the best way.

Mr. Cohen, like Mr. Hudson, remarks concerning the geometry of the structure. It is justly remarked by Mr. Cohen that in the paper no attempt is made to show the applicability of the analysis presented to economic conditions. The writer thinks that any such attempt would be fruitless. Economical conditions, it may be said, vary almost in each and every instance, and any attempt at standardization would not be serious.

The correction suggested by Professor Mylrea concerning rib-shortening is proper. This error was an oversight on the part of the writer.

The writer fully agrees with Professor Schachenmeier concerning the convenience of having a combination of a graphical and analytical method. The method of the ellipse of elasticity, although presented in a purely graphical way, can nevertheless be checked at each step by simple computations, as has been shown.

Most of Mr. Whitney's remarks and objections find their answer in the results shown in the first part of this closing discussion. He seems to have stopped at the names and titles of books dealing with this theory; for instance, he cites the name of M. Sejourné as one having dealt with this theory as applied to arch design. Had he gone further, he would have found that Paul Sejourné acknowledges that the method he explains has been taken from Guidi's work.

DISTRIBUTION OF REINFORCING STEEL IN CONCRETE BEAMS AND SLABS

Discussion*

By Boyd S. Myers, M. Am. Soc. C. E.†

Boyd S. Myers,‡ M. Am. Soc. C. E. (by letter).§—The discussion shows a wide diversity of opinion on this very important subject.

The purpose of the paper was to lead to a definite approval or rejection of the principles discussed. If those principles are correct, they should be used more generally by engineers on account of the economy of material secured; and if they are radically in error, evidently they should be discarded.

Hundreds and probably thousands of structures have been built in the United States, for which the designs were based on the assumption that the steel in the top over the supports need not exceed $\frac{W L^2}{24}$, or one-half the amount usually placed in the bottom at mid-span. These structures have given years of satisfactory service and show no visible signs of distress or weakness. This fact, however, does not necessarily indicate that the assumption is theoretically correct. Many of these designs were executed by men receiving their technical training in the best schools of the country, and their practical training with some large reinforcing bar company. Probably the reduction in the total tonnage of steel for a given project was one of the reasons for the assumption. Undoubtedly, if it is a radical departure from the true theory, there should be failures on record, with causes readily traceable to a weakness in the design. Faulty design has been the cause of relatively few failures of concrete structures.

The theoretical distribution of the moments in fully restrained concrete beams is well shown in Mr. Larson's discussion.‖ If the net section of the beam is used in computing the moments of inertia, the concrete on the tension side of the neutral axis being ignored, the results should give the correct distribution of the moments, and, consequently, the correct distribution of the reinforcing steel. A balanced section of the beam should be used in computing the moments of inertia. A flange just wide enough to develop the tension steel should be used with both steel and concrete working at their maximum

* Discussion on the paper by Boyd S. Myers, M. Am. Soc. C. E., continued from March, 1927, *Proceedings*.

† Author's closure.

‡ Cons. Structural Engr. (Brown & Myers), Oklahoma City, Okla.

§ Received by the Secretary, April 7, 1927.

‖ *Proceedings*, Am. Soc. C. E., February, 1927, Papers and Discussions, p. 275.

design stresses. The top steel at the supports will vary from 0.65 to 0.80 of that usually placed at mid-span for the ordinary case. Evidently, it would be more nearly correct to design interior spans for a moment of about $\frac{W L^2}{14}$ and to place the same area of steel in the top over the supports as at the bottom at mid-span.

The writer believes that the discussions have been of considerable value. He is fully convinced of the need of a comprehensive series of actual tests to link up in a better way theory and practice in concrete design.

PRODUCING CONCRETE OF UNIFORM QUALITY

Discussion*

By William M. Hall, M. Am. Soc. C. E.

William M. Hall,† M. Am. Soc. C. E. (by letter).‡—Uniform quality in the production of concrete is a subject that has probably received as much attention by engineers during the past few years as any other presented before the Society. It has been under discussion with more or less activity since the commencement, about 1892, of monolithic concrete construction on the Hennepin Canal, were Col. W. L. Marshall, U. S. A., used concrete so dry as to give no slump, and on the Chicago Canal where the late Isham Randolph, M. Am. Soc. C. E., Chief Engineer, used concrete wet enough to give a slump of 9 or 10 in. Hence, it hardly appears creditable to the profession that after such a long time it should be subject to such a general indictment as given§ by George F. Swain, Past-President, Am. Soc. C. E. However, it is notorious that a considerable percentage of the great volume of concrete constructed in the United States is believed to be justly subject to such criticism.

The author stresses the fact that for uniformity it is essential to have unvarying absolute quantities of the several ingredients, including water; that such a mixture would be uniform also in strength and easy of control.

Of the highly desired qualities of concrete in most concrete structures the one quality in which uniformity is most desirable is its strength. As stated by the author, a uniform mix is the basis of uniform concrete; but it is the basis only. The lack of uniformity in the materials is a basic condition preceding the mix, which affects the uniformity of the concrete. There is a lack of uniformity in the strength of the cement, in which it is not unusual to find a variation from 20% to as high as 50% in 7-day briquettes and nearly as great a variation in the 28-day briquettes. Such variations are found in many brands of cement.

With other conditions uniform, such variations in the strength of the cement will produce a corresponding variation in the resulting concrete. If the percentage of resulting variation in the strength of concrete is not as great as that in the briquettes made from the same cement, it will probably be close to it. Therefore, it appears most desirable to cure that defect in cement. Until it is cured, it appears impracticable to prevent the variation in the strength of concrete produced therefrom.

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* Discussion on the paper by Roderick B. Young, Esq., continued from March, 1927, *Proceedings*.

† Parkersburg, W. Va.

‡ Received by the Secretary, March 14, 1927.

§ *Engineering News-Record*, December 23, 1927.

Another condition affecting uniformity of strength is the great variation in both the fine and the coarse aggregate, in such localities as the sand and gravel bars in the Ohio River between Pittsburgh, Pa., at its head, and Cairo, Ill., at its mouth, 967 miles. These bars are producing material in large quantities, and will probably continue such production for centuries. It is believed that this variation in the grading of gravel at many such localities, all other conditions being equal, will produce as much as 5 to 20% variation in the strength of the concrete. Although this variation in grading is objectionable, the advisability of incurring the expense of correcting it by artificial re-grading is questionable. It is probable that more than 90% of the concrete made within 50 miles of the Ohio River is of gravel and sand from the river, and no one has attempted to re-grade it.

Possibly the condition of next importance in producing uniformity in concrete (and possibly of even greater importance than the two others named) is the lack of intelligent, well-trained, and enthusiastic workmen at the mixer and in the forms, who are actually executing all the details of keeping check on the proportioning, measuring, mixing, placing, and curing described by the author. Although more care is now being given to this training than a few years ago, it appears that even to approximate uniform quality of concrete all the men engaged in doing the work should be trained in the details of the methods of control relating to their respective jobs.

The author states that, "It is not the individual low or high test that is important* * * *". The writer does not agree with this conclusion as to low tests, but thinks, on the contrary, that a single batch of concrete which averages below the minimum strength requirement is objectionable. Such deficiency is, of course, much more objectionable under some conditions, or in some members, than in others. It appears that the test, or series of tests, should indicate that a section through any member of a concrete structure of any kind has an average strength per square inch equal to the minimum strength for which it is designed. Until such degree of perfection is reached or closely approximated, it is probable that concrete as a building material will be subject to criticism.

The writer believes that the day is near when a minimum strength for concrete can be specified and obtained with less than 1% of the batches falling below the minimum, and with an average strength of all the concrete in any monolith, or in any two consecutive batches, above the minimum. After the minimum strength is attained or exceeded by all the batches, uniformity as to the variation between the highest and lowest strength batches is not of vital importance to the safety of the structure, and is of minor importance to the owner. However, the effort to obtain uniformity in strength is desirable and is a goal which should be sought by the manufacturers of cement as well as by the makers of concrete.

The writer wishes to express his appreciation to the author for his interesting paper and for introducing this important question.

WATER-RATIO SPECIFICATION FOR CONCRETE

Discussion[*]

By F. R. McMillan, M. Am. Soc. C. E.,
and Stanton Walker, Assoc. M. Am. Soc. C. E.[†]

F. R. McMillan,[‡] M. Am. Soc. C. E., and Stanton Walker,[§] Assoc. M. Am. Soc. C. E. (by letter).[‖]—The discussion of the "water-ratio specification" has brought out a number of features which add to the interest of the subject and contribute information of permanent value. In practically all the discussion the evident intent of the contributor is the same as that of the writers. The writers believe that some of the points were covered in the original discussion, while others require some further amplification to avoid confusion. The amount of interest shown in the paper is most gratifying.

Mr. Coutlee[¶] emphasizes the important point that the absorption of the aggregate is of considerable importance in determining the net quantity of mixing water. It frequently occurs also that concrete is improved in quality by the leakage of water from the forms, loss through evaporation, etc., during placing or shortly thereafter. It must be remembered that it is the water-cement-ratio in the mass as it is solidified in the forms that actually determines the potential strength of the concrete. In the specification accompanying the original paper it was recognized that the quantity of water absorbed in a period of 30 min. may be deducted.

Mr. Woodle[¶] points out that a table of proportions of materials for different water-ratios and gradings of aggregate would be desirable. The writers recognize the convenience of such a table, but feel that it would defeat, to a great extent, one of the aims of the specification—that of flexibility in the selection of materials and proportions. He also suggests that the specification would have been more complete if it had covered the use of admixtures. This specification was not intended to cover all the special applications of concrete, but rather to cover the basic factors pertaining to proportioning to meet definite strength requirements in the usual case.

The writers recognize the point made by Mr. Woodle of the desirability of a more satisfactory test for workability than now exists. Admittedly, the

[*] Discussion on the paper by F. R. McMillan, M. Am. Soc. C. E., and Stanton Walker, Assoc. M. Am. Soc. C. E., continued from April, 1927, *Proceedings*.

[†] Authors' closure.

[‡] Director of Research, Portland Cement Assoc., Chicago, Ill.

[§] Director, Eng. and Research Div., National Sand and Gravel Assoc., Washington, D. C.

[‖] Received by the Secretary, March 26, 1927.

[¶] *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1871.

slump test is an imperfect measure of workability. This is especially true in a comparison of mixtures of widely different nature; for example, a 1:3:6 mix with a 5-in. slump would not be at all comparable in ease of placement to a 1:1.5:3 mix with the same slump. However, in the absence of a more satisfactory method, the slump test was included in this specification to mark the limits of consistency that would avoid, on the one hand, mixes too dry to puddle properly and, on the other hand, those so wet that segregation would be inevitable. The slump test finds its greatest usefulness for those combinations of fine to coarse aggregate that are more readily placeable, such as are recommended in the specification and the paper. Most of the objections developed from the use of this test have arisen where an effort has been made to hold the consistency to a very narrow range of slump, or to an exact slump, and where it has been used as a method of controlling the quantity of mixing water. However elusive that intangible quality of workability may be, lack of workability, on the other hand, is neither elusive nor intangible, but is readily recognized.

Mr. Watson* has made valuable and comprehensive comments on the different clauses of the specification. While in entire agreement with many of the points he raises, the writers feel that some of the questions may have arisen in his mind through their failure to outline in more detail the scope of the paper and specification.

His comment on the differences in strengths in Fig. 1† and Table 1‡ for the same water-cement ratios may be explained on that basis. The two sets of data represent a span of six to eight years in the history of the laboratory and include changes in technique as well as differences in materials. The tabular values are from recent tests and are typical of the higher results now commonly being obtained, both in the laboratory and in the field. Neither set of data has been corrected for absorption of water by the aggregates. If the later data are thus corrected, as is now being recommended, the margin of difference over Abrams' original curve is considerably narrowed, so that the original curve remains a good although somewhat more conservative basis for design.

Mr. Watson points out that some of the discrepancies in the data of Table 1 can be explained by the greater absorption where larger quantities of aggregate are used; and that aggregates which have a high absorption need the stronger mortar resulting from the reduction in water-ratio due to absorption. It is felt that Mr. Watson's point is well taken in extreme cases.

With reference to the differences in strength in Table 4,§ these can be explained readily by the greater loss in water during moulding from the richer and, consequently, wetter mixes. For example, in Column (1) of Table 4 the 4 300-lb. concrete had a slump of 11 in.—a very wet mix, which

* *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 2030.

† *Loc. cit.*, September, 1926, Papers and Discussions, p. 1408.

‡ *Loc. cit.*, p. 1409.

§ *Loc. cit.*, December, 1926, Papers and Discussions, p. 2030.

would loose water readily—while that having a strength of 3 560 lb. had a slump of only 2.2 in.—a plastic, cohesive mix.

The writers agree with Mr. Watson that durable concrete is the ultimate aim for all structures and that in the case of concrete exposed to the elements, durability is largely dependent on impermeability. They are not familiar with the data to which Mr. Watson refers regarding the advantages of fine sands from the viewpoint of impermeability. On the contrary, from their study of such data as are available, they are convinced that it is the proportion of uncombined water in the cement paste that determines the permeability of concrete, provided it was of the proper plasticity when placed. The evidence on this question seems quite convincing; and a logical deduction is that, in the usual nominal mixes, a coarse, well-graded sand would produce concrete of lower permeability than a finer one, due to its lower water requirements. The writers are in thorough agreement with Mr. Watson's statement that the condition of exposure of the concrete should be taken fully into account in the selection of proportions.

Mr. Watson has interpreted the clause of the specification fixing the water-cement ratio for the different strength classifications to be a specification of strength requirement. Such was not the intent; in fact, the writers pointed out that the time was not ripe for such a specification and gave in some detail the objections to it. The purpose of indicating the different classes on the basis of strength (and this was not made clear in the paper), was to meet the requirements of the Chicago Building Code, which are stated in such a way that it may be interpreted either on the basis of strength or on that of arbitrary proportions. The designation of class of concrete by letter, as Mr. Watson advocates, would have been better under other circumstances.

Mr. Watson's suggestion of narrower limits for the measurement of moisture in the aggregate is quite in accord with the writers' views. In this first specification it seemed desirable not to be too rigid and, therefore, the allowance of 2% was included. There should be no difficulty in meeting the more rigid requirement suggested by him.

He points out certain of the unsatisfactory features of the slump test, but falls into the error of assuming that it was the intent of the specification to use this test as a control of the quantity of mixing water. The slump test was used only to mark the limits of the consistency to avoid both the extremely dry and over-wet mixtures.

The section on tests of concrete to which Mr. Watson refers is admittedly not a fundamental feature of the specification. It was included to provide for the tests desired to obtain a check on this new method of controlling concrete. The results of the tests had no bearing on the provisions of the contract, except that the owners reserved the right to change the water-ratio, with proper compensation to the contractor, should the tests show this to be desirable.

Mr. Watson points out the desirability of a wider range in grading of fine aggregate than that indicated in the specification. This is undoubtedly desirable. It should be remembered, however, that the specification under discussion was written with Chicago aggregates in mind.

The additional time of mixing that he recommends would undoubtedly be desirable in many instances. Likewise, a recommendation of a minimum temperature of 50° Fahr. during the early hardening period of concrete is an improvement over the value stated in the specification. It is believed, however, that the maximum temperature of 90° Fahr. for the concrete as it leaves the mixer is unnecessarily low. The only information in the possession of the writers indicates that the limit of 120° Fahr. specified, does not approach a dangerous value.

Mr. Whitney's discussion* refers principally to the difficulty of estimating quantities of materials under this specification. Usually, this is the first objection raised to it, but one that the writers feel to be less serious than may be appreciated at first thought. As was pointed out in the paper, a few simple preliminary tests with the materials to be used will give all the desired information and with much greater accuracy than the common tables of quantities used for estimating with arbitrary proportions.

Mr. Boyden† raises the question as to which of the two water-ratio strength equations were used as a basis for the design. In this case the original equation of Professor Abrams, $S = \frac{14\ 000}{7^x}$, was used. Mr. Boyden is correct in stating that the slump test has been a useful device in improving the quality of concrete and that much better control is obtained by its use than by ordinary methods of procedure. It is the writers' point, however, that the best use of the slump test is in fixing the limits of consistency, and not as a means for controlling the water.

Mr. Perry's‡ experience with the use of the water-ratio specification is interesting and, as far as the writers are aware, represents the first instance in which this method has been applied to pavement construction. His use, however, of an approximate method of correcting for the water in the aggregate on the basis of an arbitrary mix does not seem to be as satisfactory as the straightforward method of making the correction on the basis of the actual quantities of materials used. In fact, important errors may result from the use of such an approximation.

Mr. Ahler's discussion§ would seem to be pointed toward a specification based on strength. The writers have pointed out the objection to a strength specification for general use in the present state of the art. It is recognized that in the hands of organizations experienced in these control methods, as is that of Mr. Ahlers, the strength specification is entirely feasible. The practice of establishing a job curve for the water-ratio strength relation, is a distinct

* *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 2034.

† *Loc. cit.*, January, 1927, Papers and Discussions, p. 98.

‡ *Loc. cit.*, p. 99.

§ *Loc. cit.*, March, 1927, Papers and Discussions, p. 488.

step toward eliminating variations due to differences in job conditions, materials, etc.

Mr. Merriman* has made the point that the proper design of concrete mixtures is only one of the steps in securing the desired end—permanent construction. The writers are in full accord with this statement. The point which he makes of the importance of keeping the water within the mass during the early curing period should not be overlooked. It is the loss of water during hardening that interrupts the chemical reactions so necessary to the building up of a dense internal structure that will resist the penetration of moisture and the beginning of disintegration. The significance of the design of concrete mixtures by the water-cement ratio goes beyond the mere fixing of a strength relation for standardized conditions of curing and testing; the quantity of mixing water is the basic factor in the production of impermeable concrete, for it is the uncombined water in the mass that fixes the degree of porosity of the concrete. This uncombined water is a function of the original quantity used in mixing and the extent to which the chemical reactions are completed.

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 572.

UNIT STRESSES IN STRUCTURAL MATERIALS

A SYMPOSIUM

Discussion*

BY MESSRS. JOHN TUCKER, JR., F. E. TURNEAURE,† D. B. STEINMAN,† AND J. A. NEWLIN.†

JOHN TUCKER, JR., ESQ.‡ (by letter).§—Mr. Steinman admits the absence of a "mathematical basis"|| for working stress calculations, and the Special Committee on Stresses in Structural Steel admits¶ being unable "to discover a method of rational calculation" of working stresses. That the method of obtaining working stress values is unsatisfactory may be gained from the discordant values of 12 000, 14 000, 15 000, and 16 000 lb. for maximum permissible steel column stresses, recommended by four bodies of eminent engineers of the United States within a period of five years, without substantially different data as bases. A rational method must and can be developed. Mathematical statistics affords the method of analysis, and has been suggested.** All that is now required is the perfection of details of application, and the accumulation of relevant data for analysis by this method.

Mr. Steinman, starts with the long current practice of "16 000 lb. per sq. in., the value established by rule-of-thumb", citing instances in which this value has been increased to as much as 25 000 lb. per sq. in. in design, and 26 000 lb. per sq. in. in rating existing bridges. In substance, he contends that, in the light of experience—certainly a dependable guide—the stresses should be increased. Thus far the exposition is faultless, but the exact amount of the increase is still a matter for conjecture. If a maximum of 26 000 lb. per sq. in. has been safely used, then why may not this value be accepted as the standard? The majority of engineers will not be too conservative to desire an increase in the 16·000 lb. value, but undoubtedly the vast majority would be against setting the figure at 26 000 lb. per sq. in. The determination of questions by majority vote may be politically sound, but it is out of place in the field of engineering.

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* Discussion on the Symposium on Unit Stresses in Structural Materials, continued from March, 1927, *Proceedings*.

† Authors' closures.

‡ Washington, D. C.

§ Received by the Secretary, March 1, 1927.

|| *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1434.

¶ *Loc. cit.*, March, 1925, Papers and Discussions, p. 396.

** *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), pp. 1126 and 1181.

The 50% margin of safety quoted by Mr. Steinman and also by the Special Committee on Steel Columns and Struts is, of course, perfectly accidental, and without any rational foundation. This value is a by-product, the result of the difference between what is considered by good engineering judgment to be the safe stress of the material and the strength of the material to be reasonably expected. To be consistent with his desire to eliminate extraneous variables, Mr. Steinman should eliminate this 50% margin of safety. The loading of the structure would then have to be computed so that at the very maximum it would produce stresses in the steel of the exact magnitude of the safe working stresses.

This procedure is especially desirable for buildings, where the comparatively large dead load is known with considerable precision, where the maximum live load can be computed with satisfaction, and where the wind load system can be liberally designed for emergencies of extremely high winds. The use of this suggested stress without a margin of safety will not require more accurate computations nor tend to a less safe structure. If it is not economical to determine the loading with precision, then it should be assumed large enough to cover any possible case. All uncertainties are thrown into the magnitude of the total superimposed loading, and the degree of refinement desired in the design will fix the accuracy with which the actual maximum loading must be predetermined.

A stress for compression lower than for tension does not appear logical. Examination of all tension and compression coupons reported by the Special Committee on Steel Columns and Struts shows the average of the tension and of the compression "useful limit" (yield point values not given) to differ only 1 per cent. While it is true that tension specimens with drilled or punched holes will have a greater strength on the net section through the hole, the difference is not sufficient to warrant any such difference in stress as that indicated by the various committees.

Safe stress analysis based on the mill test values for yield point, is weak and most insecure, as those familiar with such test results will concede. The usual mill test is run with such high speed and such inaccuracy that it is not to be accepted as the basis of engineering design.

Experience is a firm foundation; it is an exceedingly long, tedious, and costly process; nevertheless, it must be partly relied upon, for the experimental data on which to base analyses are incomplete, and, in some instances, misleading. Thus the results of rotating-beam endurance tests on steel of approximately structural grade appear in part to be devoid of significance, for they indicate* that several million reversals of stress well beyond the yield point will be withstood before failure, whereas it is known positively that one application of the yield point stress will cause the failure of a column.

The endurance limit of structural steel elements is an unknown quantity. For steel under one loading the influence of holes and discontinuities happily appears to be of a nature to increase the ultimate strength of the material in the net cross-section, due to the pronounced ductility of structural

* *Bulletin No. 124*, Univ. of Illinois Eng. Experiment Station, p. 77.

steel at the yield point and beyond. Endurance tests on flat specimens with an included hole show* that the material in the net section through the hole has the same endurance limit as similar material without a hole. In the rotating beam specimen with the hole perpendicular to the axis, the maximum stress computed by the usual beam formula for the net section through the hole gives a value equal to the endurance stress computed from the specimen without the hole. In general, discontinuities, if they have any effect, appear to increase the strength.

The ultimate stress under endurance tests, and the effect of high local stresses at holes and other discontinuities, both indispensable factors in working stress determinations, were not broached in the Symposium. Experience with existing steel structures must be relied on to fill the unknowns in the endurance tests, and from it the yield point must be selected as the basis for safe stress determinations.

The elimination of all variables external to the structural element, more especially those of uncertainty, as Mr. Steinman strongly recommends, is even more strongly urged by the writer. It is unquestionably the first, and a most important, step to take in the development of a rational method for the analysis of working stresses. Closely allied with this attitude is the elimination of stresses to be permitted by specifications as indefinite as "plastic mass concrete" which have been critized by Dean Turneaure.†

The rational method of working stress determination should result in a formula expressing the working stress in terms of coupon test results. It is only reasonable to assume that the requirements for structural steel will not remain constant, but probably will be slowly increased. As an example, results of tests have shown‡ that very heavy H-sections (14-in., 287½-lb.) can be fabricated with ultimate compressive strengths of 38 000 to 44 000 lb. per sq. in., whereas previous tests on heavy 8-in., 91-lb. H-sections developed only 24 000 lb. per sq. in., and 14-in., 287½-lb. H-sections only 24 000 to 28 000 lb. per sq. in. Surely this increase in strength is deserving a concomitant increase in working stress.

Table 4 shows the mean strengths of the several types of columns, together with the coefficients of variation and skew, statistical parameters of value in determining the reliable strength, and the computed safe stress for each column type. The safe stresses are computed by the method given by the writer in his paper entitled "Reinforced Concrete Columns".§ There being several different column lengths, the coefficients could not be computed from the mean of the group of like columns, and the variation from the best representative "length-strength" line was used as the datum for computing the individual variations.

It is not practical to use a different safe stress value for each different type of column, nor is it plausible, in view of the considerable difference in the coefficients of variation for the different types of columns, computed from

* *Bulletin No. 152*, Univ. of Illinois Eng. Experiment Station, p. 33.

† *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1426.

‡ *Technical Paper 328*, U. S. Bureau of Standards, p. 68.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1074.

a number that should be sufficient to give a much smaller variation, to assume that duplicate tests and computations on the groups of nine columns would give identical safe stresses.

TABLE 4.—Steel Column Ultimate Strengths and Safe Stresses, Together with Dispersion Parameters

Type.	Column No.*	Number of columns.	Area (approximate), in square inches.	Coefficient of skew.	Coefficient of variation.	Ultimate column stress,† in pounds per square inch.	Computed safe stress, in pounds per square inch.†	
8-in. H-beam...	32 lb.	9	9.15	+0.51	3.5	37 750	30 800	
	34 lb.	9	10.0	+0.09	1.9	33 660	30 400	
	62 lb.	9	17.8	−0.36	2.8	35 300	30 100	
	91 lb.	9	27.4	+0.67	4.2	25 300	19 700	
Mean..........	.....	36	.....	+0.23	3.1	......	......	
Light sections.	1	9	11.25	+0.39	2.5	33 400	29 000	
	2	9	10.0	+0.45	4.1	34 400	27 000	
	3	9	8.7	−0.11	2.5	34 700	30 100	
	4	9	11.7	−0.55	3.0	37 300	31 400	25 200‡
	6	9	13.7	−0.63	2.6	31 800	27 500	
	8	9	11.1	−0.08	2.5	35 500	30 900	
	10	9	10.8	± 0	2.5	34 900	30 300	
Mean..........	.....	63	.....	−0.07	2.8	34 570	29 500	
Heavy sections	1*A*	9	22.2	−0.50	2.1	29 900	26 600	
	2*A*	9	17.0	+0.68	2.2	32 800	29 000	
	3*A*	9	16.4	+1.06	3.8	30 400	24 300	
	4*A*	9	16.9	+0.75	2.8	28 900	24 800	20 750‡
	6*A*	9	23.7	+0.50	8.0	30 300	22 400	
	8*A*	9	25.7	−1.70	6.1	32 500	22 200	
	10*A*	9	18.6	−0.02	2.2	31 300	27 700	
Mean..........	.....	63	.....	+0.11	3.5	30 870	25 300	
Univ. of Illinois, *Bulletin No. 56*..	.....	10	13.0	+0.22	3.0	28 890	24 300	

* Designation in original paper, *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.

† For $\frac{l}{r} = 50$.

‡ For method of computation, see text.

There is no one unique method to pursue in determining the safe stress from the group of columns. From the group of safe stresses for each type of column, the reliable value of 9 400 lb. per sq. in. for a slenderness ratio of 50 was determined. This result is interpreted to mean that if an infinite variety of column types were possible, then no one selected at random would have a safe stress of less than 9 400 lb. per sq. in.

However, there are not, an unlimited, but an indefinite, number of types. Rather than attempt to define the limiting number, it was decided to pursue the method that follows. The safe stresses for the light and heavy types of column have been determined from the average ultimate column strength (for the seven sub-groups) and from the average coefficient of variation of the seven sub-types. These results, given in Table 4, show, in agreement with the ultimate strength, the superiority of the light, over the heavy, column.

It is to be noted that the steel in all columns passed the specifications of a 30 000-lb. yield point. It is unfortunate that the characteristics of the steel from which the columns were fabricated was not determined with precision from test coupons, for the stresses just determined could then be used as a criterion in conjunction with tests of future coupons to determine the stresses to be allowed in the steel from which the coupons were taken.

Concrete.—While the Building Code Committee of the U. S. Department of Commerce* specifies concrete strengths on the classification basis of the designations "plastic mass concrete" and "concrete mixed moderately wet", and the Committee gives the slump limits for these classifications, Dean Turneaure's criticism† in this connection seems to be fully justified, especially as these slump limits appear more in the nature of suggestions, and not of rigorous restrictions. The Code is also inconsistent, giving for "plastic mass concrete" a "slump of 1 in. to 3 in." on page 5, and "not more than 3 to 5 in", on page 18. The writer would go further and eliminate the "very wet concrete" (slump of 10 in., or more), as a slump of not more than 8 in. will give a concrete of sufficient workability to be applied for any purpose. If this slump without the addition of admixtures is not productive of a concrete of sufficient workability, the engineer should resort to the use of admixtures for this purpose.

Little can be added to Mr. Lindau's excellent presentation‡ other than additional data. The strength variation§ is undoubtedly the most important single factor influencing the value of the working stress for concrete. This variation, all factors influencing the strength being kept constant, is a function of the size of the specimen; the larger the specimen, the smaller the coefficient of variation. This phenomenon has been theoretically predicted and experimentally verified, and is of unquestioned importance in design. This law of the variation apparently does not hold for the built-up steel column, for reasons not yet wholly known.

The statistical analysis of test beams will undoubtedly reveal a considerable possible increase in the steel tension reinforcement, as indicated by Mr. Lindau.‖ His contention that the "present working tensile stresses * * * are too conservative", due partly to the neglect of the tensile strength of the concrete, must be tempered with an important qualification. In the critical section of a simple beam the tension concrete cracks at a stress in the steel of approximately 3 600 to 4 500 lb. per sq. in., and in the region of maximum moment the portion of the load borne by the tension concrete at the working load on the beam, is negligible. Mr. Lindau's theorem is valid, of course, when applied to the Theorem of Three Moments up to the loading that causes the first tension crack in the concrete at any point. In this connection the assistance of the concrete in resisting tensile stresses becomes of importance, since by increasing the moment of inertia of beams and slabs at those portions

* "Recommended Building Code Requirements for Working Stresses in Building Materials," released December 20, 1926.

† *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1425.

‡ *Loc. cit.*, p. 1428.

§ *Loc. cit.*, p. 1429.

‖ *Loc. cit.*, p. 1431.

that are not cracked, it causes the deflections and stresses to be appreciably less than those based on computations that neglect the tensile resistance of the concrete.* The cracking of the concrete produces a discontinuity in the moment of inertia of the cross-section making the problem exceedingly difficult of a mathematical solution. It would appear that this subject is worthy of considerable further analytical study.

Timber.—Mr. Newlin† considers all factors concerning working stresses but pursues none of them to a logical conclusion. From the data on Sitka spruce a safe stress may be computed, in place of the method used by him in reducing the value by one-quarter to cover the variation in strength of the clear wood, a procedure with apparently no foundation whatever. The frequency curve of Fig. 1‡ is of a complicated nature, being a compound curve not to be classified in any of the Pearson types. A graphical approximation by extrapolation indicates a reliable stress of 2 400 lb. per sq. in. Reducing this value by nine-sixteenths and three-fourths, Mr. Newlin's fatigue and defect constants, the working stress value of 1 010 lb. per sq. in. results, comparing with the recommended value of 1 100 lb. and the computed value of 1 820 lb. given by him. It is to be noted that the value of 1 010 lb. just derived is applicable to the specimens of the size used in the tests from which the value was derived.

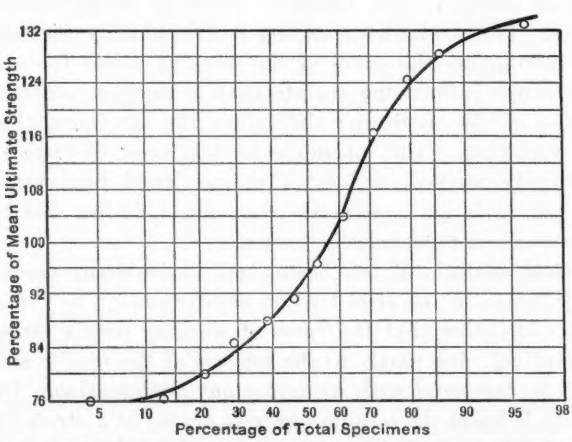

FIG. 5.—TOTAL FREQUENCY CURVE—DOUGLAS FIR BEAM STRENGTHS.

The most vital factor has been entirely neglected, namely, the difference of the strength variation with the size of the specimen. Whereas the variation apparently follows a definite law for concrete, and is independent of the size of the specimen in built-up structural steel columns, it is considerably more complicated in timber beams. This is partly due to the tendency of the timber of the larger sizes to check along the neutral axis, the locus of the maximum shear. This tendency works toward the formation of two superim-

* *Proceedings*, Am. Concrete Inst., Vol. 17 (1921), p. 415.

† *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1436.

‡ *Loc. cit.*, p. 1437.

posed beams each one-half the depth of the original beam. The strengths of large beams will tend to group themselves about two values—the strength of the beam as a whole, and the strength of the same beam when divided into two parts along the neutral axis. This latter strength will be one-half the former.

This phenomenon is indicated in Fig. 5 giving the total frequency curve for twelve specimens of Douglas fir, beams that had been in service eleven years.* To compare with the test results on the large beams, forty-eight 3 by 3 by 40-in. beams cut from the larger beams after test show a practically straight frequency curve, or a normal error distribution of the ultimate strengths. These examples are given to show the possibilities of an extensive and comparatively complete analysis on sufficient test specimens.

F. E. TURNEAURE,† M. AM. SOC. C. E. (by letter).‡—The discussion shows that there is not a wide divergence of opinion in regard to the subject of working stresses for steel. There is also evident a feeling that accuracy and reliability in design have increased considerably in recent years, but that specifications should still be reasonably conservative.

The writer can hardly agree with Mr. Whitney§ that the subject of code enforcement should not be considered in preparing specifications as general as those under discussion. Officers responsible for code enforcement are, as a matter of fact, co-operating in the manufacture of a product, particularly in the case of reinforced concrete. The final product in the building is dependent, to a considerable extent, on the degree of supervision and inspection; and it is this final product that must be considered in laying down rules for design. In the case of a steel structure, the effect of good or poor inspection is not so great as in the case of reinforced concrete, and it is for this reason particularly that the writer is inclined to be more conservative in regard to the latter material.

Mr. Tucker‖ proposes to apply the theory of probabilities to a considerable extent in determining strength values. This method of analysis may be useful where the variations in values are wholly due to uncontrollable variations in the product, and where some law could possibly be established that would represent probable deviation from the mean; but in such a structure, for example, a column, the writer believes that there is still a long way to go before exhausting the possibilities of separating the variables and reducing the controllable variations. To attempt to apply the theory of probabilities to such a complex case would seem to be misleading and without much value.

D. B. STEINMAN,¶ M. AM. SOC. C. E. (by letter).**—In reviewing the discussions of the paper on "Unit Stresses in Structural Steel (for Buildings)",††

* *Bulletin No. 41*, Univ. of Illinois Eng. Experiment Station.

† Cons. Engr.; Dean, Coll. of Mechanics and Eng., Univ. of Wisconsin, Madison, Wis.

‡ Received by the Secretary, April 4, 1927.

§ *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 2040.

‖ See p. 967.

¶ Cons. Engr., New York, N. Y.

** Received by the Secretary, April 13, 1927.

†† *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1432.

the writer has been pleased to note the practically unanimous agreement with his thesis that a working stress higher than the old value of 16 000 lb. per sq. in. is now justified. Apparently, the only difference of opinion is the amount of increase advisable for immediate recommendation and official authorization. Practically no one hesitates to go to a working stress of 18 000 lb. per sq. in., and some are ready to go all the way to 20 000 lb. as advocated by the writer.

Some of the discussions emphasize the fact that there still remain unknown elements affecting the safety of a structure: Variations in the material are pointed out; imperfect designing is mentioned; and the desirability of further tests and research is suggested. The writer concedes these limitations without altering his conclusions. If these variations and deficiencies did not obtain, a working stress close to the yield point would be justified. If the technique of design were perfect, and if knowledge of loads, materials, and structural behavior were complete, the writer would not stop at advocating a unit stress of 20 000 lb. per sq. in.; under such conditions, an ultimate working stress of 30 000 lb. per sq. in. could be used with justification.

The essential point of the argument is that, although all the unknowns have not been eliminated, they have been substantially reduced with the improvements in structural art and theory during the past generation. This reduction in the unknown and variable factors justifies a reduction in the corresponding margin of safety between working stress and elastic limit. If 16 000 lb. per sq. in. was a safe working stress a generation ago—and experience has so proved it—a higher unit stress is safe to-day. If some hesitate to make the immediate advance to 20 000 lb. per sq. in., the working stress of 18 000 lb. per sq. in. acceptable to the more conservative should be adopted for the present. It is a step in the right direction.

J. A. Newlin,* M. Am. Soc. C. E. (by letter)†—In discussing timber stresses, Mr. Tucker states‡ that "the most vital factor has been entirely neglected" that is, the checking of large timbers along the neutral axis and the likelihood of failure by shear. Although the title might indicate otherwise the paper aimed to explain the method used in arriving at a safe stress in the extreme fiber in bending only. It is true that this other phase of the subject is very important.

The U. S. Forest Products Laboratory has investigated the shearing in large beams and its relation to the stress developed in the standard shear specimens, and has also made a careful study of columns both large (up to 12 by 12 in. by 24 ft.) and small. It is almost inevitable that large beams will check along the neutral axis. The checking usually increases after the timber is in place. There are, however, compensating factors, that the worst checking usually occurs in straight-grained beams having a high quality of clear wood, and that the two beams resulting from a horizontal shear failure will almost never be seriously checked along their neutral axes so that when the original beam has failed in shear at a dangerously low load it will generally

* With U. S. Forest Service, Forest Products Laboratory, Univ. of Wisconsin, Madison, Wis.

† Received by the Secretary April 4, 1927.

‡ See p. 972.

be found that the two resulting beams will carry a load considerably in excess of that which caused the original failure. These two beams are, as a rule, amply strong to carry the design load safely for long periods of time, but they lack materially in stiffness.

Failure in shear is, however, a common cause for the removal of railway bridge stringers and also of considerable worry and expense. It does not seem to be feasible to limit shakes and checks in large timbers as closely as knots or cross-grain; therefore, to obtain a safe stress from tests of small clear specimens it is necessary to use a factor one and one-half times as great for shear as for stress in the extreme fiber.

EXPERIMENTAL DEFORMATION OF A CYLINDRICAL ARCHED DAM

Discussion*

By W. H. R. Nimmo, Assoc. M. Am. Soc. C. E.

W. H. R. Nimmo,† Assoc. M. Am. Soc. C. E. (by letter).‡—The experiments described in this paper generally confirm the analysis of the problem given by Mr. Smith in his previous paper§ but there is considerable variation between the several experimental values found for E_0 and E_1. The values of E_0 (Young's modulus) determined from the direct tensile tests should be 143 and 123 lb. per sq. in., giving a mean of 133 lb. per sq. in., or 9.35 kg. per sq. cm. The values of 151 and 139 lb. per sq. in. given in the paper are presumably in error and do not agree with the value of 125 lb. per sq. in. obtained from a repetition of the experiments.

The formulas given by the author for H and V may be derived from the theory of thin arches, as usually developed for pipes and sewers, and involve the assumptions that the axial length of the ring is short and that the thickness, t, is so small in comparison with the radius that the effect of curvature on the moment of resistance of the beam may be neglected. On these assumptions it can also be shown that there are: A circumferential thrust at the crown, $P_T = -\frac{w\,a}{2}$; a circumferential thrust at the ends of the horizontal diameter, $P_s = \frac{\pi\,w\,a}{2}$; a circumferential thrust at the bottom, $P_B = \frac{w\,a}{2}$; a circumferential moment at the crown, $M_T \doteq \frac{w\,a^2}{2}$; a circumferential moment at the ends of the horizontal diameter, $M_s = -0.5708\,w\,a^2$; and, a circumferential moment at the bottom, $M_B = 1.50\,w\,a^2$; in which, a positive value of P denotes compression and a positive moment, M, is one that produces tension at the inner surface of the ring. In the case of a ring of unit axial length, the extreme fiber stress at the inner surface at the crown due to the moment, M_T, will be,

$$f_T = -\frac{6\,M_T}{t^2}$$

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* Discussion on the paper by B. A. Smith, M. Am. Soc. C. E., continued from April, 1927, *Proceedings*.

† Chf. Draftsman, Main Roads Comm., Brisbane, Queensland, Australia.

‡ Received by the Secretary, March 22, 1927.

§ "Arched Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2042.

(the negative sign denoting tension) and the corresponding unit deformation will be,

$$\frac{f_T}{E_0} = -\frac{6\,M_T}{E_0\,t^2}$$

Owing to the effect of Poisson's ratio, the circumferential moments will give rise to axial deformations of varying sign at different parts of the ring, the surface of which will, therefore, undergo considerable warping. In the case of a long cylinder, however, such warping could not take place to any great extent and the axial deformations would be largely neutralized by axial moments, the magnitude of which would be, approximately, m times the circumferential moment at the corresponding part of the cylinder (m denoting Poisson's ratio). The axial moment at the crown would give rise in its turn to a unit circumferential deformation of the extreme fiber at the inner surface of $\frac{6\,m^2\,M_T}{E_0\,t^2}$ and, therefore, the net unit deformation of the extreme fiber at the crown at the inner surface would be,

$$\frac{6\,m^2\,M_T}{E_0\,t^2} - \frac{6\,M_T}{E_0\,t^2} = \frac{6\,M_T}{t^2}\left(\frac{m^2-1}{E_0}\right)$$

Since the extreme fiber stress at the inner surface at the crown is $-\frac{6\,M_T}{t^2}$, the modulus of elasticity for the circumferential stresses in the cylinder is,

$$E_c = -\frac{6\,M_T}{t^2} \div \frac{6\,M_T}{t^2}\left(\frac{m^2-1}{E_0}\right) = \frac{E_0}{1-m^2}$$

For rubber, the value of m is about $\frac{1}{2}$ and, therefore, $E_c = \frac{4}{3}\,E_0$. If $E_0 = 9.35$ kg. per sq. cm., then the value of E_c becomes 12.5 kg. per sq. cm.

The experimental value of $E_1 = 13.6$ kg. per sq. cm. is about 9% in excess of the value deduced for E_c and the difference may be due partly to an error in the adopted value of $m = \frac{1}{2}$. It may also be largely due to the effect of curved beam action, which is neglected in deducing the formulas for H and V.

If the value of $E_c = 12.5$ kg. per sq. cm. be adopted for both vertical and horizontal stresses in the cylinder when the latter is filled with mercury, then the values for u given in Table 7 are obtained. It will be noted that, for the case of the base simply supported, there is a closer agreement between the computed and observed values of u in Table 7 than obtains in Table 3.* For the case of the base encastrée, the results, for values of z exceeding 3, are better than those in Table 4;† but for the lower part of the cylinder the results are not so good. In the latter case, since there is a displacement of more than $\frac{1}{8}$ in. at a height of 1 in. above the base, considerable curvature exists in vertical radial planes and, therefore, any formulas that neglect the effect of curvature will not give correct results.

* *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1600.
† *Loc. cit.*, p. 1601.

TABLE 7.—VALUES OF RADIAL DISPLACEMENTS, u, IN CENTIMETERS.

(E = 177 lb. per sq. in. = 12.5 kg. per sq. cm.)

z, in inches.	BASE SIMPLY SUPPORTED.		BASE ENCASTREE.	
	Computed.	Observed.	Computed.	Observed.
0	0.000	−0.01	0.000	0.00
⅛	......	......	0.009	0.00
¼	......	......	0.032	0.07
⅜	......	......	0.068	0.16
½	......	......	0.103	0.20
⅝	......	......	0.146	0.24
¾	......	......	0.187	0.31
⅞	......	......	0.229	0.36
1	0.465	0.47	0.278	0.37
2	0.509	0.53	0.450	0.50
3	0.378	0.40	0.379	0.47
4	0.238	0.24	0.244	0.24
5	0.113	0.10	0.115	0.11
6	0.001	−0.95	−0.001	0.00

In the beam experiments described in the paper, the deflection amounts to more than one-twelfth of the span. Consequently, the value of E_1 = 172 lb. per sq. in., deduced from the ordinary straight beam formula, $D = \frac{W\,l^3}{48\,E\,I}$, is not comparable with the value of E_0 obtained from the direct tensile tests.

THE DESIGN, CONSTRUCTION, AND OPERATION OF A SMALL SEWAGE DISPOSAL PLANT

Discussion*

By Franklin Hudson, Jr., Jun. Am. Soc. C. E.†

Franklin Hudson, Jr.,‡ Jun. Am. Soc. C. E. (by letter).§—The writer is appreciative to those members who contributed useful and interesting information, and criticism in their discussions.

The wash troughs described by Mr. Thackwell|| for carrying the floating scum from the gas-vent area of the Imhoff tank to the sludge drying beds should be of definite value where foaming troubles are anticipated or where constant attendance and operation are not possible. It is a question, however, whether or not such wash troughs would be used to advantage in the ordinary installation for the small municipality. In all probability, the average operator would tend to use these troughs continually rather than to stir and break up the unstable scum in order that all undigested solids might be allowed to sink to the sludge level again to "ripen" properly.

Mr. Thackwell's description of the small sewage disposal plant is very interesting, because the average sanitary engineer meets with small plant problems in ever increasing numbers.

The remarks by Mr. Benham¶ relative to the lack of health laws, especially in Oklahoma, rigidly requiring a systematic operation of sewage treatment plants, are timely. At present, threats of lawsuits and nuisance complaints by the property owners living below the outlet of a plant seem to be the usual method of compelling the municipality to maintain its treatment plant. That such conditions will eventually be remedied is evident from the fact that an increasing number of those citizens most interested are continually bringing pressure to bear on the law-makers at each session of the Legislature.

The writer agrees with Mr. Enslow** that even the smallest plants need daily attention. However, in the case of the usual small town, with a population of less than 5 000, the always present factor of cost must be taken into consideration, and it is certainly true that thorough attention once each week, even with a possible waste of chlorine, will be less expensive than daily operation. During the period that the plant at Stroud, Okla., was operated by the writer, the following schedule of operation was carried out, and on turning

* Discussion of the paper by Franklin Hudson, Jr., Jun. Am. Soc. C. E., continued from February, 1927, *Proceedings*.

† Author's closure.

‡ Secy.-Treas., Benham Eng. Co., Kansas City, Mo.

§ Received by the Secretary, April 6, 1927.

|| *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 2047.

¶ *Loc. cit.*, January, 1927, Papers and Discussions, p. 119.

** *Loc. cit.*, February, 1927, Papers and Discussions, p. 305.

the plant over to the city authorities a copy of this schedule was framed and hung on the wall of the chlorinator building and an additional copy filed with the Water and Sewer Superintendent:

"OPERATION SCHEDULE
SEWAGE DISPOSAL PLANT, STROUD, OKLAHOMA,
PLANT TO BE ATTENDED NOT LESS THAN ONCE EACH WEEK

"WEEKLY ATTENTION

"Imhoff Tank.—Thoroughly clean entrance screen with rake and empty all screenings into the incinerator.

"Skim all floating débris from the entrance to the sedimentation chamber and also stir up settlings here so that they will enter sedimentation chamber.

"Carefully scrape sloping walls of sedimentation chamber with downward movement to force all clinging particles into sludge compartment.

"Break up all scum, hard crusts and floating matter in gas vent with rake so that same will settle.

"Gauge flow of sewage from weir at outlet trough.

"Sprinkling Filter.—Remove all nozzles and clean and allow one dose of sewage to discharge before replacing. To remove nozzles it is only necessary to turn them to the left and withdraw as they are of the lock type.

"Chlorinator.—See that amount of chlorine as indicated by the instrument is sufficient to properly treat the sewage in accordance with the following:

"Depth of Sewage over Effluent Weir at Imhoff Tank.	Pounds of Chlorine per 24 Hours. (3.5 Parts per Million.)
¼ in.	0.6
½ "	1.8
¾ "	3.3
1 "	5.0
1¼ "	6.9

"Follow the instructions as outlined for chlorinator instrument in the book of instructions furnished by the Wallace and Tiernan Company.

"ONCE EACH MONTH

"Sprinkling Filter.—Using pipe wrench remove all *nozzle seats* that are along east side of filter and allow sewage to discharge once, then remove remaining nozzle seats and allow sewage to again discharge. Clean, oil, and replace.

"As the sewage is discharging the first time the operator should use the rake and stir contents of dosing tank so that any settlings will pass out on to the filter.

"EVERY THREE MONTHS

"Sludge Bed.—Remove dried sludge from bed and replace any surface sand that has been removed.

"Imhoff Tank.—Open sludge valves, one at a time, and allow ripe sludge to flow out on to sludge drying bed. Use one bed for this.

"Secondary Settling Basin.—Using small diaphragm pump remove settled sludge from basin by pumping on to small sludge bed.

"Make an inspection trip to the plant after every rain storm."

It is admitted that the foregoing instructions are only for manual control of the plant but, nevertheless, as long as they are conscientiously followed a satisfactory sewage effluent will result.

TOWN PLANNING AND ITS RELATIONS TO THE PROFESSIONS INVOLVED

Discussion*

By John Nolen, M. Am. Soc. C. E.†

John Nolen,‡ M. Am. Soc. C. E. (by letter).§—Each of the discussors has made comment of value, which becomes evident to any one who will follow the discussion through. In this final summing up, the main points will be mentioned.

The subject "Town Planning and Its Relation to the Professions Involved" unquestionably calls for the training, technical skill, and experience usually spoken of as engineering, architecture, and landscape architecture. In all important work these services should be rendered, if possible, by a group of individuals. In some cases, however, it is not practical, because of cost or other reasons, to secure a complete group, and, in other instances, such a group may not be essential.

The employment of a single office, primarily that of an engineer, an architect, or a landscape architect, does not mean that only the training, skill or experience of one profession is employed, because an office properly equipped for city planning as a regular part of its work almost invariably has on its staff men of various training and experience—engineers, architects, and landscape architects.

Several definitions of city planning have been given, which are helpful as revealing various and somewhat contrasting points of view. For example, Mr. Alvord|| defines town and city planning as "those foresights in physical adjustment necessary for the fullest expression of the individual in his community life"; again, he states:

"Unity of general purpose and fitness of each co-ordinating function, with variety and harmony in detailed method, as practical opportunity is afforded, is the keynote of good town planning."

In contrast with this is the statement of Mr. Bassett.¶ He asks,

"What is town planning? By this is not meant the objects of town planning or the results of town planning. Town planning is the determination of the legal quality of land areas for public purposes."

This statement may be true enough if the limitations which Mr. Bassett has himself placed upon it are kept in mind. It is apt, however, to be con-

* Discussion on the paper by John Nolen, M. Am. Soc. C. E., continued from April, 1927, *Proceedings*.

† Author's closure.

‡ City Planner, Cambridge, Mass.

§ Received by the Secretary, April 4, 1927.

|| *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, pp. 1873–1874.

¶ *Loc. cit.*, January, 1927, Papers and Discussions, p. 121.

fusing, and does not appear to be a correct definition of "town planning". It is of the utmost importance that the planning itself should be properly done by qualified individuals before this legal quality is fixed in the land. The supreme element in city planning is that of design. The trouble to-day in existing cities is largely because the legal quality that exists in the land is not the right legal quality in the right land. Streets, for example, are wrongly located, and not of proper width. Railroads are improperly placed. There is a great lack of the necessary open spaces. Foresight has not been exercised in selecting sites for civic buildings, public schools, etc., and the wrong use or over-use of private land has been established. Mr. Bassett speaks also of the change in the use of land. This may be very desirable, for the change may represent changing requirements.

Professor Lyle,* refers to what might be called the evolution of modern city planning. He mentions the "city beautiful" idea, and the emphasis on civic centers; also the co-ordinate problems of traffic control street widening; river and harbor improvement; and railroad terminal and track changes. All this illustrates the comprehensive character of modern city planning and the necessity for a broad point of view, not only of the city itself, but of a region.

Regarding the important question of the organization of city planning, and the inter-relation of the various planning professions involved, one of the best statements was from Mr. Alvord, in connection with "the United States Housing Corporation at Washington, D. C., where town planners, architects, and engineers were closely co-ordinated in nearly one hundred housing developments." The impression made on him was, "not the separateness of these three callings, but, at bottom, the remarkable similarity."

Mr. Adams† also drew from his wide knowledge and experience in this field when he pointed out that effort should be made to bring about the needed collaboration, mentioning the British Town Planning Institute, to become a member of which a person must be a full member of the Institution of Architects, of the Institution of Civil Engineers, of the Surveyor's Institution, or a qualified lawyer.

The same idea is well brought out in the discussion of Mr. Cauchon,‡ who refers directly to the practice of the Town Planning Institute of Canada.

In conclusion, perhaps the most constructive suggestion to come out of this discussion is the final word from Mr. Adams', in which he states:

"The American City Planning Institute should be re-organized and developed on a strictly professional basis, with due regard to bringing in the architect, the engineer, the landscape architect, the lawyer, and, on a few exceptional occasions, the sociologist and others who are especially interested and who have already gained a certain standing. There are greater tasks awaiting the city planner in the future than in the past, but in order to deal with them he must be better equipped with technical knowledge and better organized."

* *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1874.

† *Loc. cit.*, December, 1926 Papers and Discussions, p. 2049.

‡ *Loc. cit.*, April, 1927, Papers and Discussions, p. 582.

HEXAGONAL PLANNING, TRAFFIC INTERCEPTER, AND ORBIT

Discussion*

By Messrs John Nolen, E. P. Goodrich, Lawrence Veiller, George B. Ford, and Noulan Cauchon.†

John Nolen,‡ M. Am. Soc. C. E.—The profession is indebted to Mr. Cauchon for the constructive thinking that his work represents, and the combination of fresh ideas related to old ideas, and new terminology for well-recognized principles in science and art, design and construction.

The one thing that is particularly significant is the presentation of new plans and technique to meet the new conditions, standards, and requirements, of the world to-day, and the necessity of recognizing that broad part of the field of planning which is not merely the patchwork and piecemeal system of doing what we can, what we must do, with existing cities.

This paper involves somewhat the by-play of planning cities according to preconceived plans. Triggs, in a book written about 1909, divided cities, under preconceived plans, into three classes which he called the spider's web, or the radial system, the rectangular or chessboard, and a combination of the two classes. In other words, he practically limited it to those classifications, and stated:

> "In nearly every case these are founded upon some geometrical figure, a square or triangle, or the favored hexagon. * * * But such schemes, though they appear well on paper, are almost impossible to carry out in reality, because they could only be carried out on a level plane, and they pay little or no regard to the natural features that are generally the prime reason of a city's location. At the same time such ideal plans have their value. The most successful form that has been yet devised is the hexagon, of which type we illustrate two examples, both having certain points to commend them. It is claimed for the hexagonal type that it permits the development of the city to the utmost that might be possible within many decades, because, with the hexagon, the great advantage of the diagonal line is secured and at the same time many intervening spaces are provided that are so much in demand in all city plans."

An interesting comment in this general field is what might be called the "bee-hive" type, in "The Spirit of the Hive", by Dallas Lore Sharp. There is also the old "roadtown" type' by Chambless, which is another scheme. The "Regional Planning Theory" by Arthur C. Comey, M. Am. Soc. C. E., is a reply to British town planning. The satellite towns program is particularly

* Discussion of the paper by Noulan Cauchon, Esq., presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926, and published in this number of *Proceedings*.

† Author's closure.

‡ City Planner, Cambridge, Mass.

well illustrated in plans drawn by Messrs. Raymond Unwin, Robert Whitten, and others.

Arterial highways and hexagonal planning is the combination and recognition of the traffic problem in connection with hexagonal planning. Mr. Cauchon advocates a principle of through intercepting arteries, free from local traffic, as opposed to the present practice of enlarging existing arteries; also, that new residential subdivisions should be on the hexagonal pattern and obtain their through-traffic service by intercepting arteries.

There is the fundamental question of whether a city can be planned as a project with any sense of completeness, in its whole area and with regard to its whole purpose. It must be recognized that the biological principle of growth is in the nature of a city and that it must carry with it opportunities for reasonable expansion and change of function, and that is why such planning comes to be rather on the side of art than of science. In attempting to make city planning mechanical and to standardize it, except as a principle of organization, limitations and trouble are immediately invited because that is not the nature of either the city or of human life. Paradoxically as it may seem, the most permanent thing is change. A scheme which is not elastic enough to have this dynamic element of change, will soon show its limitations. This does not mean, however, that every design of a railroad, or building, or bridge, or city does not, in some way, try to bring within the human limits of possibility the forecasting of conditions over which they may have control.

E. P. Goodrich,* M. Am. Soc. C. E.—Mr. Cauchon has pointed out with a great deal of accuracy most of the fundamental principles which must underlie a scientific design of any community. With all due regard to those who are the predecessors in the working out of the art of the planning, it is the science of city planning that is going to do more in the future to create the better living conditions, the cheaper traffic operations, and the easier running of this big machine than has been possible under the development of the art so far as it has gone to date. This is one of a series of suggestions in the development of that technique. It is a very interesting start, and the mind naturally reverts to how the present gridiron type, which is almost universal, can possibly be adapted to produce the same general effect that Mr. Cauchon has suggested. For example, his introduction of an interior play area into the center of a block is obviously possible in a rectangular system exactly in the same way in which he has developed it in the hexagonal scheme. Even his playground is simply the working out of such a scheme as was actually developed in Washington, D. C., for the improvement of one particular block in its interior. Thus, there is no real virtue in the hexagonal shape as far as that is concerned.

Substantially this system, as Mr. Cauchon indicated, was developed in 1803 in Detroit, Mich., around the central part of the city, but as he has suggested it has been abandoned and traffic requirements have dictated for economic reasons the cutting through of various other direct routes so as to

* Cons. Engr., New York, N. Y.

not break up the traffic as has occurred heretofore, but to get it away as fast as possible. It has been found that this system in Detroit actually congests traffic more than the direct route system which has been created in other parts of the community and is now being superimposed on that rather extra length system in the down-town section. Furthermore, the people who live in this particular area are the only ones who know how to get out of it. A foreigner, a non-resident, always gets lost in that part of Detroit, and even if he is from the outlying sections he cannot go through it with any degree of certainty that he is going to "arrive". While theoretically there is an advantage, practically there is a great disadvantage in that type of street layout.

Mr. Cauchon believes that the radial and the circumferential idea is the one that should apply in the design of a city structure. If to that general statement were added the condition that it applies to the natural nuclei of the community, it would be generally accepted. An examination of traffic actually moving in different communities, for example, in Cincinnati, Ohio, or Norfolk, Va., where special studies were made, shows that if areas are selected with some reference to traffic origin, such units create traffic in certain proportions depending on the residential character, the density of the population, and the use of the streets for business and industry, and that there is a desire on the part of the people in one particular unit to go to each of the other units in proportion to the inverse ratio of the distance between those different sections. There should then be in any proper design, a system of what might be called spider webs connecting the nuclei as directly as possible. That will take the traffic from one point to another rapidly. It will reduce the cost as far as traffic is concerned. It will reduce the frontage which must be devoted, in part, to the use of through traffic. Between these various spider webs, possibly the hexagonal system might be applied to advantage. It is also possible to make varieties of interior arrangements as irregular as one chooses so as to provide the same friction to through movement in those sections. Many designs have been submitted and actually laid down on the ground in various sections, St. Louis, Mo., for example, where the specific object has been to prevent the use of such streets by through traffic. There is small tendency to move through such developments, however, if the nuclei are connected as directly as possible. It is entirely feasible in the connection of these various sectors or elements to make a connection by one device or another from the intermediate interior streets to the main thoroughfares.

Mr. Cauchon has very strongly pointed out the need of sunlight in modern city development, and has suggested that the hexagon will provide the best possible arrangement for the street system so as to secure the utmost of sunlight in rooms of the buildings on his frontage. All agree that sunlight is important; in fact, that it is the most important element possible, not only in residential planning, but in industrial and even in commercial planning; that it will be in the future more and more appreciated; and that the future scientific design of cities will probably take into account sunlight planning to a greater extent than ever before.

The speaker must, however, dissent from the contention that the hexagon is the best shape. Fig. 6 is a sun dial designed for the latitude of New York. It gives the length and direction of a shadow at any day and hour. For any community of a different latitude the curves would be somewhat modified. The north point is vertical. At noon, the shadow would go from the center of the circle to any one of the different curved lines, depending on the time of the year. At the winter solstice the shadow would be from the center of the circle to the upper of the curved lines. At any time of the day the shadow would go from the center to the point on the proper one of the curves which is intersected by the straight line giving the hour. With this sun dial one can lay down a street system and see whether he can get sun into the rooms facing on certain streets.

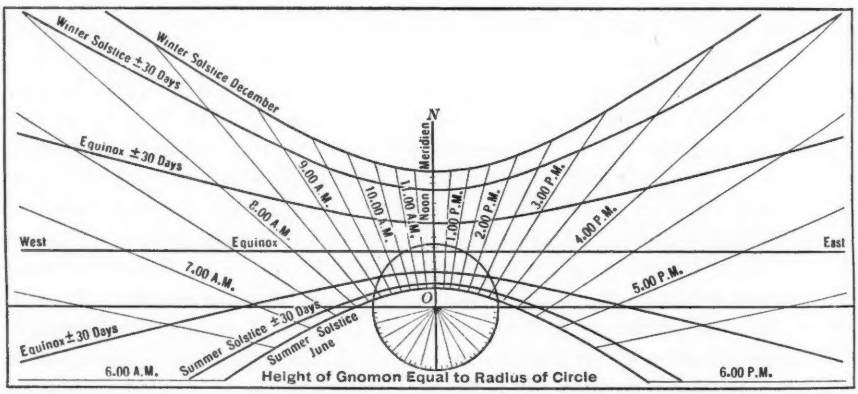

FIG. 6.—LENGTH AND AZIMUTH OF SHADOW OF UNIT GNOMON CAST BY SUN AT ANY HOUR OF DAY AND AT ANY TIME OF YEAR, LATITUDE 40° 45′ NORTH.

Using this sun dial, studies have been made of the period over which the sun would penetrate a window facing in each direction on the shortest day of the year. After studying the matter, it was felt that a 30-min. interval of penetration was the minimum which should be adopted. A 1-min. penetration is insufficient to secure any curative effect of sunlight of any consequence, or any of the other benefits which direct sunlight penetrating a room has been shown to have from a medical standpoint.

There is so little advantage in getting the sunlight in the street as compared with the need of getting it into the room, that the simple fact of the sun shining up the street from time to time, is not as great as 10% compared with getting it into the dwelling itself Now it can be shown by the use of the sun dial and the penetration figures, that a properly oriented rectangular street system is much superior to the hexagonal scheme from a sunlight standpoint.

Mr. Cauchon has suggested the need of intercepters, which are essential; but if the intercepters are to maintain the hexagonal routing there applies to them, as also to the street system of the small hexagon, the question as to distance and the economics of traffic.

In Fig. 4,* if a person wanted to move from a point at the lower end of one hexagon directly east, obviously there is an extra distance in the ratio of 100 to 75 compared with a straight line. The straight line is 75% of the distance along the hexagonal. Likewise, if one wanted to move south there would be the ratio of 100 to 87. If one wanted to move in a direction across the hexagon parallel with one of its sloping sides, and in place thereof had to go around a rectangular block, the ratio is as 102 by the rectangle to 100 by the hexagon. If a person wanted to go south 30° east, the hexagon compared with the rectangle is as 100 to 138. In a 45° direction the ratio is 103 rectangular to 100 hexagonal. In other words, in going around the circle, if one happens to strike the direct route of the rectangular system of straight streets, about 25% has been gained. If one wants to go diagonally, the hexagonal system is shorter by between 2 and 3% up to 38% in one instance. So that, generally speaking, it is disadvantageous to move by the hexagonal route compared with the rectangular one by a margin of 10 or 15%, which, because of the fact that the cost of operation of the system is so much greater than the interest on the investment, will overcome the benefit of the shorter length of streets with reference to area which obviously applies to the hexagonal system.

There is the further difficulty, that some day, whether it be on the rectangular or on the hexagonal system, traffic regulation will be arranged so as to be continuous at all times on every street. It will not be a stop-and-start system, but it will have continuous motion. That can be worked out in two ways with reference to the rectangle, but when it comes to the hexagon, it seems impossible.

With the rectangular type, traffic can be in one direction on one thoroughfare and in the other direction on another, moving in blocks so co-ordinated or regulated that traffic can move continuously on all the thoroughfares; or, similarly, by designing streets in pairs or in certain other schemes, all traffic will move on north and south thoroughfares continuously at all times, and on the east and west thoroughfares block by block. If that scheme is worked out and the north and south thoroughfares are designed to be three times the capacity of the east and west streets, it will be found possible to move in an east and west direction continuously.

The whole problem of street layout design is one of economics of administration, based on such a scheme as will produce the best possible acquisition of sunlight and traffic feasibility in any community. As between the rectangular and the hexagonal systems, it seems at the moment that those two elements are in favor of the rectangle rather than the hexagon. in spite of the brilliant suggestion which Mr. Cauchon has made.

LAWRENCE VEILLER,† ESQ.—Mr. Goodrich has subconsciously applied the author's plan to an existing, built-up community, and especially a great city like New York or Chicago, and Mr. Cauchon, at the very outset and in all presentations of his plan heretofore, has absolutely disclaimed any such idea. He has said this was a scheme for the development of a residential

* See p. 789.

† Secy. and Director, National Housing Assoc., New York, N. Y.

community on the new parts of an existing community. He has also tied to it the scheme of intercepter highways.

Mr. Goodrich's presentation does not detract in any way from the scheme, but on the contrary seems to re-endorse the arguments as presented. All the questions that he raises as to these different foci of interests and attractions apply to an existing community, because when a new town is built they do not exist. There will be other foci, but Mr. Cauchon's scheme takes care of them. So that argument falls to the ground as far as it relates to the new community, but it does apply to existing communities because there are tendencies already developed that cannot be changed.

Again, with regard to the question of regulating traffic which Mr. Goodrich rightly points out, using the two diagrams of collision or confusion points: There will not be any necessity to regulate traffic if these residential communities are developed. There will be no police signal systems, for there will not be any such plans. The whole thing is scientifically planned to discourage that. Mr. Goodrich was no doubt thinking in terms of cities like New York which have the gridiron system where traffic has to be regulated. With regard to his control of traffic, that could well be the subject of a paper to be delivered at some future time.

GEORGE B. FORD,* ESQ.—Two questions arise about the extremely interesting subject which Mr. Cauchon has presented.

In the first place, with regard to the shape and size of individual lots: Would not a more varied scheme—a combination of winding roads and straight roads—allow fully as much space per individual householder and at the same time give more interesting shapes to the lots and more variety than the hexagonal scheme could possibly give?

In the second place, in looking at the general hexagonal plan one gets the impression that the total amount of space used in streets in the hexagonal scheme plus the superimposed thoroughfares is actually greater than the total amount of space that would be used in a rectangular scheme even with diagonal arteries superimposed on it, particularly on account of the number of half-blocks, and quarter-blocks that are left where the hexagonal scheme joins into the thoroughfare spider web, and on to the rectangular street layout.

NOULAN CAUCHON,† ESQ.—In regard to the size and shape of the lots: The basis of all planning is the technique of sociology. The standard lot which is used, is 100 ft. in depth, and varies in width. A great deal of it is needlessly put into backyards only partly used. The number of attempts to make collective playgrounds of these backyards are evidence of the fact that they are little needed in their present form. In the hexagonal block the backyards, or those parts of the backyards, that can be spared for recreational purposes, are assembled in a more advantageous way than can be done in the longer rectangular block.

As to uniformity, the hexagon is not limited to any exact size, neither is is essential that it should equal some particular rectangular block. If a rea-

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sonable area for a unit could be agreed upon, say, 1 000 sq. ft. per individual, this, transposed into a lot 125 ft. deep, would produce more backyard. The difference between these two in respect to the shape of the lot is the purely social advantage in having the surplus backyards aggregated in the hexagonal form. In zoning, about 30% of the lot is allowed to be covered by buildings. It is immaterial to the lessor whether the remainder of that lot, 70%, is a backyard or a fine playground.

As to the artistic side, it is not necessary that this should be carried out everywhere. There is no reason why certain areas should not be treated in a landscape manner, with winding roads, especially down hillsides where there are good engineering reasons for having a winding road. It is simply suggested as a basic pattern for residential districts, not for business districts. In the business sections, right angles are retained in order to facilitate grade separation and traffic control.

In regard to the other point Mr. Ford raised, the imposition of these large traffic arteries certainly takes up some of that 10%, but to advantage. The 10% advantage is an advantage purely in the length of the street. A field of hexagons would discourage through cumulative traffic, and residential streets could then be reduced to possibly 30 or 40 ft. in width. Farther away from the intercepters they do not have any great load of traffic to carry; there is practically a country road seclusion. What is taken off the width of residential streets can be added tc the main traffic areas, and still be several per cent. ahead of the present scheme, with a better traffic arrangement.

In regard to the question of housing: New York has a State Commission on housing, and its report (Document No. 11 925), reveals officially that practically two-thirds of the working population, below a certain level, cannot be provided with housing for themselves on an economic basis. In other words, the cost of living is so great that the laborer cannot provide himself with decent living conditions, nor will the costs of money allow of its provision in the State of New York. There is something radically wrong. That report is considered one of the finest public documents ever published, as it reveals the condition to be faced. It is an economic condition which, if it cannot be helped, simply means degradation to the class of people which cannot earn enough to house itself or be housed decently. Such conditions have to be changed or the race will die out. One of the advantages of the immigration laws is that they are going to raise the standard of those coming into the country, but, if a condition is to be maintained whereby the population in the country cannot be self-sustaining on the basis of decent living, it means the annihilation of the race.

There is a cost, and a justifiable one, of survival; a person or a group of people cannot be allowed to have the power of throwing the cost over on the remainder of the community; thus pauperizing them to live at the cost of the other people. Referring to the question of slums, the following illustration is applicable: the cannibal cooks and eats an individual right off while he is fat; the commercial cannibal imposes living conditions which gradually disintegrates the human frame; he makes his victims skin and bones before he

eats them; he practically lives on their blood. It is a matter of degree, but that is exactly what is happening, and the State of New York or any other State, or Province in Canada, which has a growth that is going to destroy the life of the community must have such malign growth extracted—to begin by the abatement of congestion.

The diagram (Fig. 4*) illustrates the point that there should be no centralized concentration. The rectangular system on a limited scale is not harmful. One of the psychological troubles with love for the unlimited gridiron is that city planners are accustomed to it and think every man should have a thoroughfare straight where he wants to go. An attempt was made here to design something which would dissuade the public from the use of residential streets as through thoroughfares. There is no reason why every one should require to go straight through to somebody who lives five or six miles away. It would be easier to drop in on a general through street which is wide enough for all traffic. Evidently one cannot design something that will adapt itself in all shapes and forms for everything that is wanted. That is the trouble with the present plan, the assumption of illimitable convertibility as desirable. In Fig. 6,† Mr. Goodrich is evidently thinking in terms of through traffic.

The intercepters happen to develop that way. There are six ways out of the hexagon and it gives that much greater flexibility and discretion. It fits. The intercepter can be put in any of these directions. Keep out of mind that it is a fixed diagonal; through traffic like surface traffic should adopt a path of least resistance, that is simply to be applied by the engineer who has experience to fit it to topography. It is not a case of creating a plan, but of finding what is the best way out. The plan is not being imposed upon the topography. If through traffic can be dissuaded from going indiscriminately through the hexagonal field and keep it in the intercepters, there is no need for great width of streets for there is no cumulative traffic. An inquiry: What is the proper width for a street? It somewhat depends on how long it is to be. If it is 10 miles long the street might be sufficient if it were 50 ft. wide; if it were shorter it might be necessary to have it 60 or 150 ft. wide; according to the density of the adjacent and cumulative population and traffic sheds.

Then, there is the question of sunlight. It is not claimed that divergent streets are as good of themselves as north and south streets, but it is claimed that the top and bottom of the hexagon is a great deal better than an east and west street, and you cannot have them all north and south. The generalized condition of the divergent and short length streets, coupled with the openings of the hexagon, allows of a greater sun penetration on the average throughout the entire system. There is a big building being erected in Ottawa, a type of which bankers and insurance companies are fond—with deep walls and big pillars—which faces north. If the winter sun ever shines on the north side it will never be into the windows. That is not a rational way of planning. A style of architecture which is beautiful in the south because it is a protection against the sun is not beautiful in the north where it lacks

* See p. 789.
† See p. 986.

the sun. The ideal is only ideal in the measure of its fitness for purpose, and a thing that is not ideal is not beautiful.

In Ottawa the only decent streets were built one hundred years ago by Colonel Bey, the Engineer Founder of the city. A statue to Colonel Bey is to be erected by the Dominion and it will be the first statue erected in Canada to an engineer. The town planning he did was this: He simply visioned wide streets; he made a number of streets 132 ft. wide. They are the only decent streets in the city.

GARBAGE DISPOSAL

A SYMPOSIUM

Discussion*

By Messrs. Samuel A. Greeley† and John V. Lewis.†

Samuel A. Greeley,‡ M. Am. Soc. C. E. (by letter).§—The trend of the discussions emphasizes again the difficulties in properly relating the engineering and administrative aspects. Mr. Eddy‖ justly comments on the importance of operation and on the fact that sufficient funds for satisfactory operation are not always available. It is true, as he states, that careful operation is necessary to avoid unsatisfactory results with any method of disposal.

Mr. Tribus' comments¶ bring clearly to mind the large economic loss involved in the abandonment of the Staten Island reduction plant; which, in some way, should have been avoided by a sound correlation of engineering and administration.

Mr. Jackson** brings out the tendency in many hog-feeding projects of shifting the disagreeable features of disposal to a different political subdivision. Whereas this usually isolates the hazards of offensive conditions, there is little justification for burdening outlying districts with a nuisance from the garbage collected in other political subdivisions without compensation. As he states, the supervision of the municipal departments could well be extended to apply to the hog farms.

Mr. Bassets' remarks†† illustrate the need of properly adjusting the method of disposal to its environment. The fact that a particular method is satisfactory under one set of atmospheric and collection conditions, does not mean that it will be satisfactory under different conditions.

Mr. Sperry‡‡ in commenting on the disposal for Grand Rapids, Mich., does not show any provision for sanitary control of the hog farm, thus throwing the burden on the outside territory.

Mr. Allen's data§§ picture the enormous quantities involved. They further illustrate that the cheapest disposal is not always the one to be adopted and

* Discussion on the Symposium on Garbage Disposal continued from December, 1926, *Proceedings*.

† Authors' closures.

‡ (Pearse, Greeley & Hansen), Chicago, Ill.

§ Received by the Secretary, April 13, 1927.

‖ *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 2051.

¶ *Loc. cit.*, p. 2052.

** *Loc. cit.*, p. 2053.

†† *Loc. cit.*, p. 2054.

‡‡ *Loc. cit.*, November, 1926, Papers and Discussions, p. 1876.

§§ *Loc. cit.*, p. 1877.

that more cost is justified in order to gain a more sanitary disposal. The neighborhood's objection, which he mentions, of having garbage plants located in its midst, is more especially true of large plants. A careful engineering survey should be made to determine the best sites so as to justify the locations selected. Mr. Allen also states the difficulties encountered in proper separation, giving rise to nuisance on ash dumps of decomposing garbage. This illustrates the need for sustained effort to instruct householders in the proper treatment of garbage, ashes, and other refuse.

JOHN V. LEWIS,* ESQ. (by letter).†—This paper has tended to create an interest in the Cobwell System and the results to be obtained from it, rather than a wide discussion of the paper itself. The fact that the system is a comparatively new one; that it has not been used extensively by municipalities to date; and that only three such plants are now in actual operation, probably accounts for the limited comments.

In their discussions, Messrs. Eddy‡ and Tribus‡ have mentioned the major factor which spells success or failure for the Cobwell System from a sanitary standpoint, namely, proper maintenance and upkeep. While reasonable care and attention are essential for the satisfactory operation of any type of refuse disposal plant, the complicated and intricate system requires more constant observation and upkeep. This matter also has a direct bearing on the question of economy or net cost of operation. To an equal extent, however, the market value of the by-products will decide the question of what type of disposal plant a municipality should install.

Unfortunately, the market for the by-products of reduction plants has been very unstable during the past few years, tending to bring the mixed refuse destructor type of plant into greater use in the United States. The net cost of high-temperature incineration as outlined, for instance, by Mr. J. A. Burnett, in his paper§ on "High-Temperature Incineration at Toronto, Ontario, Canada", must prove more interesting to the average public works official than the same operating cost for a reduction plant with the tankage fertilizer and grease market at its present level.

* San. Engr., in Chg. of Refuse Disposal Div., Dept. of Public Works, Rochester, N. Y.

† Received by the Secretary, April 8, 1927.

‡ *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, pp. 2051-2052.

§ *Loc. cit.*, October, 1926, Papers and Discussions, p. 1666.

THE NEW YORK STATE BARGE CANAL AND ITS OPERATION

Discussion*

By Roy G. Finch, M. Am. Soc. C. E.†

Roy G. Finch,‡ M. Am. Soc. C. E. (by letter).§—The paper was prepared primarily for the purpose of presenting the facts relating to the New York State Barge Canal. The writer is grateful for the discussion which brought out such widely divergent views as to the past, present, and future of this waterway.

Because this canal is now used to only one-tenth of its capacity, it does not necessarily follow that the engineers, who twenty-five years ago approved of a "barge" type of canal to connect the Great Lakes with tide-water, failed to analyze the problem correctly or that they "blundered" in selecting this type as best adapted to serve as the waterway connection across New York State. On the contrary, what definite proof has been presented to show the economic advantage of attempting to operate large vessels on restricted inland channels as against the plan of operating on ocean, lake, and canal the particular types of craft that can operate on each waterway most efficiently? The latter was the governing economic principle laid down in 1903, which dictated the selection of the "barge" type of canal. Up to the present time it has not had a fair chance to prove or disprove the soundness of such principle. Apparently from the discussion of General Taylor‖ he still feels that this type of canal can be made to serve adequately its original intended purpose—a feasible economic waterway connection between the Great Lakes and tide-water.

Mr. Thomson¶ refers to "two monumental blunders" in constructing the Barge Canal, namely, the introduction of the summit level at Rome and the building of fixed bridges. It seems doubtful whether tonnage has been driven away from the canal due to the existence of this summit level. If Mr. Thomson had in mind the adoption of the plan embraced in one of the earlier studies of a route for a deep waterway, it would seem that this plan had its disadvantages. It contemplated one level from Fulton, on the Oswego River, to Frankfort, on the Mohawk River, a distance of 72 miles. It called for raising the level of Oneida Lake and making a cut of 60 ft. in the vicinity of Rome.

* Discussion on the paper by Roy G. Finch, M. Am. Soc. C. E., continued from January, 1927, *Proceedings*.

† Author's closure.

‡ Cons. Engr., Albany, N. Y.

§ Received by the Secretary, April 8, 1927.

‖ *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 135.

¶ *Loc. cit.*, December, 1926, Papers and Discussions, p. 2058.

The topography and the extent to which this section of New York State has been built up and developed, would make a canal constructed on such a plan objectionable, particularly so when the available water supply is sufficient to avoid limiting the project to such a plan.

The "second error" to which Mr. Thomson refers has to do with the bridges, which have a minimum clearance of 15½ ft. above the water. He points to the Canadian canals, where the use of boats of any height is made possible by movable bridges.

It must be kept in mind that the country through which the Canadian canals are constructed is quite different as to density of population and industrial development, from the highly developed part of New York State traversed by its canal system. With any great amount of traffic, the required frequent operation of the many railroad and highway bridges might prove highly detrimental to the successful handling of the constantly increasing volume of rail and vehicular traffic in this section of the State. Furthermore, the carrying capacity of the general type of craft navigating the Canadian canals is about 2 600 tons. On the New York State Canal System, with restricted bridge clearances, a barge fleet consisting of five barges and a steamer can carry 3 600 tons. If the canal were deepened so that large self-propelled units could safely load to a draft of 11 ft. 9 in., they would then have a carrying capacity of at least 2 000 tons. This would seem to indicate that boats, properly designed and equipped and comparing favorably as to carrying capacities with the types of craft in use on the Canadian canals, could be operated successfully even with limited bridge clearances.

Professor Finch* asks three pertinent questions. The first is whether local traffic can ever be expected to justify the canal. It seems to be the consensus of opinion that the canal is adapted for packet service. Packet lines, however, have not been organized and operated to the extent of furnishing any sound basis for an estimate as to the amount of local tonnage that might be attracted to the canal. Due to the advent and mobility of the motor truck, it would seem doubtful whether local traffic in sufficient volume can be secured to "justify" the New York State Barge Canal.

His second question, "Is it not true that the only type of through traffic in sight and in sufficient volume to justify the canal is the export grain business?" can probably be answered in the affirmative with certain reservations. Grain is the commodity now moved in greatest tonnage on the canal, followed by stone, sand, and gravel, which, in turn, is followed by petroleum products. Prior to the opening of the canal, the anticipated grain movement, which is an eastbound movement, was the basis of the estimate that 70% of the total freight movement would be eastbound. It is interesting to note that in 1926 the westbound movement was about 51% of the total freight movement on the canal.

The writer believes that General Taylor in his discussion answered the third question asked by Professor Finch.

* *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 141.

The discussion by Mr. Bolton* is worthy of the most careful consideration, for he speaks not only as an engineer, but as a shipper "who is using the canal in a satisfactory way as a dependable economical means of transportation." He questions whether the deepening of the canal to 15 ft. and increasing the width in restricted sections to 110 ft. would result in placing more boats in operation. The locks were originally designed with widths of 28 ft., which was in harmony with a bottom width of channel in restricted sections (75 ft.) controlling for less than 30% of the length of the entire system. The width of the locks was subsequently increased to 45 ft., but with no corresponding increase in channel dimensions. Under existing conditions, the canal is "over-locked". The abutments of the bridges spanning restricted sections are so located that channel improvements can be carried out and not interfere with these structures. In connection with the maintenance dredging, the channel is gradually being deepened to 14 ft.

There is a wide difference of opinion among boat operators as to what is the most efficient type of operating unit. Boats having lengths of 100 ft. and widths of 20 ft. are competing with self-propelled units with lengths up to 258 ft. and widths of 42 ft. If boats of this latter type prove to be the most efficient unit, their successful operation demands a wider channel in restricted sections and the opportunity to load to the maximum depth permissible and clear the miter sills of the locks. Certainly the channel improvements suggested would not be detrimental to the operation of the smaller units and would make it possible for the larger carriers to increase their carrying capacities, which should result in increased earnings and thus make canal navigation a more attractive field for investment. Until it is demonstrated that boats can be operated at a profit, capital will continue to be reluctant to enter the canal navigation field; and a canal without an adequate number of boats will never be a success.

Mr. Bolton, in discussing a continuing canal personnel and the adoption of a definite and fixed policy on the part of the State concerning the present and future status of the operation of its waterways, has touched on a vital point so far as the future of the canals is concerned. As the result of a Constitutional amendment, a re-organized State Government started functioning on January 1, 1927, and all the activities of the State are now placed in a limited number of departments, which number cannot be increased unless a further Constitutional amendment is approved. The canals are placed under the jurisdiction of the Department of Public Works and the head of that Department is appointed by the Governor and the term of office is contemporaneous with that of the Governor. There seems to be little likelihood at this time of placing the canals under a separate department or commission.

During the past few years, certain changes have been made with a view to securing a more permanent canal-operating personnel by placing more positions under Civil Service. This should result in an increased continuity of service among those charged with the maintenance and upkeep of the canal. The effect is bound to be helpful.

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 589.

Whether justified or not, there has been a feeling that the State might "scrap" its canal system or turn it over to the Federal Government as a part of a ship canal project. Such a sentiment has retarded the development of the canal and has been detrimental to the building up of its traffic.

In the writer's opinion, the greatest stimulating influence which at this time could be brought to bear on the canal would be for the State, in no uncertain terms, to re-affirm the policy of continuing the present system; to maintain it at the highest state of efficiency; and to make changes and betterments designed to promote the best interests of navigation and thereby give to this waterway a fair chance to prove whether or not it can serve as a dependable, ample, and economic water connection between the Great Lakes and the Atlantic seaboard.

QUANTITIES OF MATERIALS AND COSTS PER SQUARE FOOT OF FLOOR FOR HIGHWAY AND ELECTRIC-RAILWAY LONG-SPAN SUSPENSION BRIDGES

Discussion*

By William G. Grove, M. Am. Soc. C. E.

William G. Grove,† M. Am. Soc. C. E.—As the author requests full discussion of the paper from all angles and as the eye-bar type of cable was eliminated from consideration after a brief reference to cables,‡ this discussion is presented for the purpose of comparing the eye-bar and wire types of cable and of attempting to show some of the possibilities of eye-bar cable construction.

In Table 1,§ five different span lengths, ranging from 1 000 to 5 000 ft., are considered, and the approximate sag ratios, that is, the ratio of the cable sag to the span length, from the assumed dimensions are as follows:

1	to	10	for	the	1 000-ft.	span
1	"	11	"	"	2 000- "	"
1	"	$11\frac{1}{2}$	"	"	3 000- "	"
1	"	12	"	"	4 000- "	"
1	"	12	"	"	5 000- "	"

For the eye-bar type of cable these ratios seem somewhat small and a more economical design would result by increasing the actual amount of cable sag for each of these five examples. However, for the sake of uniformity the discussion will be based on these ratios, for which the length of the cable is about 3% greater than the span length for the 1 000-ft. span and about 2% greater for the other spans. Also, for these ratios, the stress at the tower is about 8% greater than that at the center of the span for the 1 000-ft. span; about 7% greater for the 2 000-ft. span; and about 6% greater for the 3 000, 4 000, and 5 000-ft. spans. With the eye-bar type of cable it is therefore possible and economical to adjust the sectional area of the cable along the arc so as to provide for this variation of stress. The average area will be about 2% greater than that at the center of the span for all the span lengths given.

Assuming an eye-bar cable of any cross-section, say, 100 sq. in., at the center of the span; add 2% to allow for the average area of the cable;

* Discussion on the paper by J. A. L. Waddell, M. Am. Soc. C. E., continued from April, 1927, *Proceedings*.

† Asst. Engr., Am. Bridge Co., New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1766.

§ *Loc. cit.*, p. 1768.

add 3% to allow for the increase of the arc of the cable over the span length; and, finally, add 15% for details such as eye-bar heads and pins. Then, for a 1 000-ft. span, the cable under its own weight only, will have a unit stress of approximately 5 000 lb. per sq. in.

Allowing 2% increase for average cable area, 2% for increase of arc over span length, and 15% for detail, the unit stress, in a cable under its own weight only, will be approximately,

11 000	lb.	per	sq.	in.	for	a	2 000-ft.	span
18 000	"	"	"	"	"	"	3 000- "	"
24 000	"	"	"	"	"	"	4 000- "	"
30 000	"	"	"	"	"	"	5 000- "	"

Therefore, it is easily seen that ordinary steel (16 000 lb. per sq. in.) cannot be used for a 3 000-ft. span, because the stress in an eye-bar cable under its own weight without any truss, floor, or live load, is 18 000 lb. per sq. in., or more than is used for that grade of steel. Furthermore, heat-treated steel having an elastic limit of 50 000 lb. per sq. in., cannot be used in a span of 5 000 ft., because the stress in an eye-bar cable under its own weight, without any truss, floor, or live load, is 30 000 lb per sq. in. and only 32 000 lb. is permissible in this grade of steel.

In order to compete with steel wire for long-span suspension bridges, there has been developed the high tensile strength heat-treated eye-bar steel, having a guaranteed minimum elastic limit of 75 000 lb. per sq. in. and a guaranteed minimum ultimate strength of 105 000 lb. per sq. in. The first application of this high tensile strength heat-treated steel was in the Florianopolis Bridge in Brazil, designed by H. D. Robinson and D. B. Steinman, Members Am. Soc. C. E., and the results of the thirteen full-sized eye-bar tests on that bridge, showed an average elastic limit of approximately 85 000 lb. and an average ultimate strength of 123 000 lb. per sq. in. On that basis it was considered satisfactory to use a working stress of 50 000 lb. per sq. in.

On all wire cable suspension bridges, to the speaker's knowledge, the area of the cable is constant from tower to tower. Therefore, the stress at the tower determines the entire section so that the area of a wire cable will be 8% greater than required by the stress at the center for a 1 000-ft. span, 7% greater for a 2 000-ft. span, and 6% greater for the other three spans. On wire cables, 8% should be added for details to cover clamps and cable wrapping.

If the cost per pound of the wire cable, complete in place, including cable clamps and the small wire protective wrapping, for a 1 000-ft. span is multiplied (1) by $\frac{1.08}{1.15}$ to allow for details; (2) by $\frac{1.08}{1.02}$ to allow for the fact that the section of the wire cables is constant between towers while the eye-bar cables have varying thicknesses; and (3) by the ratio of $\frac{50\,000}{70\,000}$, the denominator being the unit stress allowed by Mr. Waddell for wire cables, there results a cost per pound which the eye-bar cable must approximate in order to compare favorably with a wire cable. The product of these factors is about 0.7. Fol-

lowing the same procedure for the other spans and using the proper factors in each case, it is found that the products of the three factors vary from two-thirds to seven-tenths.

While this method is satisfactory for the shorter spans it is not sufficiently accurate for the longer spans because it does not take into consideration the the fact that part of the unit stress in the cable is used to support the cable itself. This part is only 5 000 lb. per sq. in. for the 1000-ft. span, while it becomes 30 000 lb. for the 5 000-ft. span. The former figure is only a small percentage of the unit working stresses, while the latter figure is a large percentage, and by increasing the span a length will be reached for which the entire working stress of 50 000 lb. will be used to support the cable itself, this length being approximately 8 200 ft.

Using the unit stress ratio such as to allow for the fact that the cable must support itself, the revised products of the three factors are:

0.69	for	the	1 000-ft.	span
0.65	"	"	2 000- "	"
0.60	"	"	3 000- "	"
0.54	"	"	4 000- "	"
0.47	"	"	5 000- "	"

Assuming $0.18 per lb. (given by Mr. Waddell) as the cost of the wire cable in place, including the clamps and wire wrapping, it is found that the cost of an eye-bar cable in place should approximate:

$0.124	per	lb.	for	the	1 000-ft. span
0.117	"	"	"	"	2 000- " "
0.108	"	"	"	"	3 000- " "
0.097	"	"	"	"	4 000- " "
0.085	"	"	"	"	5 000- " "

in order for the eye-bars to compare favorably with wire; but other factors affect this comparison.

While it is true that the eye-bar cable will require somewhat heavier towers and anchorages than the wire cable, due to the fact that it contains more steel, there is another factor that offsets this increase. In the modern wire cable suspension bridges, the type of erection equipment used for the construction of the cables has been materially different from that used for the erection of the remainder of the main and side spans. In the case of an eye-bar cable suspension bridge it is possible to use the same erection equipment for the main span as that used for the erection of the eye-bars.

For the Florianopolis Bridge the entire main span and eye-bar cables were erected by the overhead method, that is, by small trolleys running on erection ropes. The trolley used for the eye-bar erection weighed 20 000 lb. and the two small trolleys used for the erection of 1 000 members of the main span, trusses, floor-beams, stringers, bottom laterals, top laterals, top struts, and sidewalks brackets, weighed only 5 000 lb. each.*

* For a complete description of the design and erection of the Florianopolis Bridge, see p. 707.

This brief discussion is presented merely for the purpose of showing that eye-bar cables using heat-treated steel having an elastic limit of 75 000 lb. per sq. in., compare favorably with wire cables for all except extremely long spans. It is not a fair comparison, however, to think of the cable alone; the entire structure in place must be considered. Furthermore, each individual bridge must be considered on its own merits. It is impossible to determine by estimate alone which type is the cheaper. There is only one sure way to find out, namely, to ask for comparative bids, as was done for the Florianopolis Bridge, where the contract price of the eye-bar cable in place was lower than the lowest bid or estimate received on either wire cable or wire rope cable in place, and the cost of the bridge was inside the estimate. Finally, the overhead trolley method of erection is safe because the Florianopolis Bridge with all its novel features was successfully erected 6 000 miles from the United States without the loss of a single human life.

THE STRESSES IN A FREE PRISMATIC ROD UNDER A SINGLE FORCE NORMAL TO ITS AXIS

Discussion*

By Henry D. Dewell, M. Am. Soc. C. E.

Henry D. Dewell,† M. Am. Soc. C. E. (by letter).‡—The interest of engineers in the subject of the stresses set up in engineering structures by earthquakes is increasing. In particular, the study of the subject by American engineers may be said to have begun after the San Francisco earthquake of 1906. This interest gradually waned in the years following 1906, but was again revived after the great Japanese earthquake in 1923, and has been growing since, the subsequent St. Lawrence, Santa Barbara, and Montana earthquakes having shown that no section of the country can truthfully be said to be immune.

The Society has a Special Committee which is diligently studying the subject of earthquakes with particular reference to their effect on engineering structures. Heretofore, and even at the present time, it is the Japanese engineers who have given most thought to the subject, and the results of their studies and conclusions have been published from time to time. Unfortunately, but a small part of this material is available in the English language. At least one treatise on the design of modern buildings to resist earthquakes has been published by Japanese engineers,§ and has now been translated by the Committee. The writer knows of no book in the English language that is comparable to this work. The Committee has also just completed the translation of a mass of pertinent data furnished by its Japanese members. In addition, it is also engaged in experimental work, having undertaken the determination, by accurate actual measurement, of the periods of natural elastic vibration of a number of buildings and other structures in San Francisco and vicinity. It is pertinent also to point out that the California Institute of Technology and Leland Stanford, Jr., University have undertaken to determine the effect of vibratory motion on structures, by means of a shaking table and models of engineering structures.

This résumé is given to emphasize what is true, but not often realized, namely, that the intense and systematic study of the causes and effects of the great natural phenomenon of earthquakes is of recent date. There is a great deal about the nature, cause, and effect of earthquakes that is not known, and there probably are phases that may never fully be understood. Nevertheless,

* Discussion on the paper by Joseph N. Le Conte, Esq., continued from April, 1927, *Proceedings*.

† Cons. Engr., San Francisco, Calif.

‡ Received by the Secretary, February 18, 1927.

§ Treatise by Dr. T. Naito.

experience and study have shown that earthquake-proof structures are not only possible, but are economically possible.

The general subject of the resistance of structures to earthquake shock may be divided into two major parts. The first is the study of the earthquake itself—its cause, the manner in which the shock is propogated through the earth's crust, and the characteristics of the earthquake wave, including period, amplitude, and direction, and consequent intensity at various distances from the origin. These points are primarily the object of the seismologists's study. The results are, obviously, of vital interest to the structural engineer. Indeed, without the seismologist's conclusions the engineer could do little, and without a basic knowledge of seismology the engineer is at a great disadvantage in attempting to design for safety against an earthquake.

The second major portion of the general subject is the finding of the stresses set up in an engineering structure by the earthquake waves, together with the proper design of the framework to resist those stresses. This division of the subject falls fundamentally in the province of the engineer, and the seismologist very properly turns over this phase of the problem to him.

The engineer is primarily interested in the effects of that class of earthquakes known as "tectonic", as distinguished from those caused by volcanoes. The tectonic earthquake is believed to be caused by a slipping along a "fault plane" of one block of the earth's crust upon another. The slip and resultant "elastic rebound" are held to send out elastic waves throughout the materials of which the earth is composed. These elastic waves, as registered at any point of the earth's surface, are complex in nature. In general, however, two types are to be distinguished, namely, longitudinal and transverse. The longitudinal waves, similar in character to sound waves in that the vibrations are in the direction of propogation, are those of compression and dilatation. The transverse waves, on the other hand, are akin to light waves, in that the vibrations are transverse to the direction of propogation.

The effect of passing through or impinging on media of varying densities in the earth's crust is to cause refraction and reflection. These phenomena, together with the interference of the waves, result in the complexity of the waves noted in the record of the seismograph.

The first earthquake waves to be recorded at any point of observation are the longitudinal waves; later, the transverse waves are recorded. Thus, it has been a relatively common experience in earthquakes to note that the shock came first from one direction, and then from a second, the two directions being frequently at right angles to each other. Also, if the focus of the earthquake was near the observer, the first effect felt may have been a sudden "shove" or "bump", followed by a rapid swinging motion. Students of seismology are not in agreement on the importance to engineering structures of the effect of the first longitudinal wave. Aside from seismographic records, which seem to be contradictory on this point, the testimony of many observers gives evidence leading to the belief that very often the collapse of some structure is coincident with the arrival of this first shock, which strikes the foundation of the building as with a blow. It is to this phase of the subject that the paper is believed to apply.

As noted in the Synopsis,* Omori, the late and eminent Japanese seismologist, observed that very rarely did tall chimneys fracture at the base. He is credited with the statement that tall free standing chimneys will always tend to break at a point about two-thirds of their height. An examination of the records of such fractured chimneys, from a study of which his conclusions were drawn, will show that, although the average location of such fractures is at a point approximately two-thirds of the height of the chimney, the variation from such average is large.

Again, as far as the writer has been able to determine, the method of computation used to find the stresses at any section of the chimney is to assume that these stresses result from a force acting at the center of gravity of that part of the chimney above the section under consideration, and equal in amount to the mass of that portion multiplied by the assumed value of the acceleration of the earthquake wave. Expressed mathematically,

$$m = \frac{W}{g} \qquad (29)$$

$$F = m a = \frac{W}{g} a \qquad (30)$$

$$M = F x = \frac{W}{g} a x \qquad (31)$$

in which,

m = mass of chimney above the horizontal cross-section under consideration.
W = weight of this mass.
x = vertical distance from this horizontal cross-section to the center of gravity of the mass above.
g = acceleration of gravity.
a = horizontal acceleration of earthquake.
F = force of earthquake acting on the portion of the chimney above the horizontal section.
M = effective moment of the force of the earthquake on the horizontal section of the chimney.

Now with the dimensions of masonry chimneys as ordinarily constructed, it may be easily shown that the weakest section of the chimney as against the bending moment, M, is at the base. In other words, if the action of the forces on the chimney is as here indicated, which has heretofore been assumed,† then all brick chimneys should fracture at the base, which conclusion is not in accord with the facts. The use of a higher acceleration in computing the bending moments at about two-thirds of the height than is used for the base, as advocated by some engineers, is certainly not a logical method of reconciling theory with observation. Evidently, this view of distribution of shears and bending moments in tall slender structures, such as chimneys, is incorrect.

* *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 3.

† For example, "Building Structures in Earthquake Countries," by Alfredo Montel; "Reinforced Concrete and Masonry Structures," by Hoole and Kinne, pp. 599, 600, 665.

Again, if the computed values of the varying section moduli of the sections of chimney from top to base be plotted to scale, that is, the vertical distances of successive cross-sections above the base being laid out as ordinates, and the corresponding section moduli be laid out properly as abscissas, and, further, if the curve of the theoretical bending moment as found by Professor Le Conte* be superimposed on the first curve (to the same vertical scale), it will be seen that the variation of the two graphs is roughly the same for the upper two-thirds portion of the chimney. Hence, after making allowances for possible sharp changes in section of chimney, instead of a uniform rate of change in section modulus from top to bottom, for the effect of unequal quality of workmanship in construction and for the known uncertain value of brick masonry in tension, the records of brick chimneys fractured by earthquakes are in accord with the distribution of shears and bending moment as deduced by the author.

The writer realizes what Professor Le Conte has pointed out, that the paper deals with a purely theoretical problem, and that the fundamental assumptions of rigidity and freedom from restraint are not realized in any engineering structure. Nevertheless, he believes that the formulas given in the paper are roughly applicable to the solution of the probable stresses due to earthquakes in certain engineering structures.

The effect of oscillation on a structure has not been covered by the author. However, he and Professor Hoskins, of Leland Stanford, Jr., University, at the suggestion of the Society's Special Committee on the Effect of Earthquakes on Engineering Structures, have investigated this problem and have solved it for the simple and ideal case of a vertical beam subjected to an oscillating force, producing simply harmonic motion of the base. The results thus proved mathematically have been demonstrated experimentally by Professor Le Conte. The resulting conclusions are remarkably simple of practical application. The determining feature is the ratio of the natural period of elastic vibration of the structure to the period of the oscillating force. Also, Dr. Naito, a Japanese engineer, has shown by a survey of buildings damaged in the Tokyo earthquake of 1923, that the damage to such buildings is apparently dependent on the ratio of their natural periods of elastic vibration to the average period of vibration of the most violent part of the shock. These further investigations are being studied by the Special Committee.

The profession should be grateful to Professor Le Conte for the results of his investigation. The writer hopes the paper will arouse interest in the general subject of the earthquake resistance of engineering structures to the end that intensive study of the subject may become more general, and that eventually certain fundamental laws may be evaluated. Until engineers, in general, are in agreement on the proper design of structures to resist earthquakes, but little progress can be expected in the matter of influencing the prospective owners of buildings and other structures to consider possible earthquake effects when planning the structures.

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* *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 15, Fig. 7.

STORAGE REQUIRED FOR THE REGULATION OF STREAM FLOW

Discussion*

By Messrs. W. W. Brush, Richard H. Gould, A. Streiff, and Sigurd Eliassen

W. W. Brush,† M. Am. Soc. C. E.—The speaker is not in a position to discuss critically the development of the curves which Mr. Sudler has shown, but wishes rather to direct attention to a closely related subject—the question as to what represents a proper amount of storage to obtain a certain yield from any stream or any group of streams.

In connection with the water supply of New York City an important problem is the determination of the safe draft each year, and in the different seasons of the year, from a water-shed with a given storage development. The safe draft should not be confused with the safe yield of a water-shed, which is considered to be a fixed amount whereas the safe draft varies with the volume of stored water and the season of the year.

The use of the term "safe yield" of a stream is regrettable. Usually, in discussing the question of the available water supply from any given development, it is said that the "safe yield" is a certain amount. The layman interprets the term "safe" in the sense that he ordinarily hears it applied and believes that if the figure given is not exceeded, there is no question as to the supply being sufficient at all times. This is not so. Since Alfred D. Flinn, M. Am. Soc. C. E., calculated the so-called "safe yield" of 336 000 000 gal. per day with the present storage in the Croton water-shed that figure has been generally adopted as the dependable yield.

In the Catskill System with the present storage development the "safe yield" has been set at between 585 000 000 and 600 000 000 gal. per day. The citizen of New York who reads such a statement believes that he can depend on 336 000 000 gal. daily from the Croton, and on, say, 600 000 000 gal. per day from the Catskill System; and that there need be no worry about drawing that quantity of water from those two sources.

The engineer responsible for the daily supply of water to New York City would be open to severe criticism if he should proceed on the theory that the so-called "safe yield" could be drawn. As is well known to engineers, the "safe yield" is the quantity of water that could be drawn daily from any source provided that no more severe drought were experienced than

* Discussion on the paper by Charles A. Sudler, M. Am. Soc. C. E., continued from April, 1927, *Proceedings*.

† Chf. Engr., Bureau of Water Supply, Dept. of Water Supply, Gas, and Electricity, New York, N. Y.

the one assumed; and provided that, on the very day the water stored in the reservoirs was entirely exhausted, the necessary rain were to fall to raise the stream flow above the rate of draft. Such a depletion of the stored water in any water supply system would be extremely hazardous.

Both Allen Hazen,* M. Am. Soc. C. E., and Mr. Sudler † have pointed out that it is much more accurate to give a percentage of the years in which a certain yield can be depended on and not to call any yield the "safe yield."

The speaker is unable to offer a satisfactory substitute for the expression, "safe yield". He believes engineers should give up the use of that term because it is misleading; and should endeavor to express more accurately to the public what is meant when it is said that a certain yield can be obtained from a certain water-shed with a certain amount of storage development.

The curves that the New York Water Department use in operating are not along the lines developed by Mr. Hazen or Mr. Sudler. Mr. Hazen's method has been used in computations to determine about what percentage of years the so-called "safe yield" could be depended on. The Department's limiting curve for draft has been obtained by a comparison between the rainfall and flow records of the Croton and Catskill water-sheds and the application of the Croton long-time record (nearly sixty years) to the Catskill streams, where a record of twenty-three years is available.

The storage required at the beginning of each month in the year to maintain a certain rate of draft is determined. The resultant curve for the entire year is plotted. If the water in storage exceeds that shown by the curve for any day in the year the rate of draft may be any amount desired until the actual storage and the storage shown by the curve coincide. Thereafter, the rate of draft cannot safely exceed the rate for which the curve was computed. By developing curves at rates of draft of 450 500 550 and 585 000 000 gal. daily for the Catskill System, an excellent control diagram is made available for the engineer in charge of operation. During 1926 the rate of draft exceeded that shown by such curves, as the Catskill supply was about to be increased by the completion of the Gilboa Dam and the creation of the Schoharie Reservoir. A very low flow for the first ten months of 1926 with the Ashokan Reservoir almost empty demonstrated the advisability of adhering to a conservative rate of draft on a surface storage system.

There are a great many other supplies to which the application of the curves given by Mr. Sudler, and those developed earlier by Mr. Hazen would show that the supply usually drawn is in excess of that which would be reasonably considered a dependable supply for the community. Engineers should do what they can to educate the public to the fact that the supply obtained from a stream is not nearly equal to the average run-off and, in many cases, is equal only to a small fraction of the average run-

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

† *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1917.

off, if a dependable supply is to be secured. At present, in discussing the future supply of New York City, there are some who very strongly argue that the average flow of a stream is the basis on which one can determine the quantity of water safely available for the city's use.

In the Croton River the average flow for one period of 18 consecutive years was about 346 000 000 gal. daily, while for the following 18-year period the average stream flow was greater by about 100 000 000 gal. daily; and still there are those who say that it is possible to take the average flow in determining what may be estimated to be the safe yield.

The majority of people believe that, because water goes over the Croton Dam, there is an available supply which is thrown away, and that the city is being mulcted in some way of a part of the water supply it should have from the money expended to develop the Croton System. They will not believe the engineer who says that it is impossible to take all the average run-off of the Croton River, and still have enough left to supply the city when the run-off is below the average. It is not the most ignorant part of the population who believe this myth but the more intelligent man.

Whether the majority of the citizens of New York will see the true picture of their water supply development is uncertain, but it is certain that a great deal of educational work should be done in eradicating from the public mind the idea that surface streams can furnish a supply equal to their average flow. Generally speaking, the so-called "safe yield" is certainly not a safe yield for even 98% of the years, and engineers should try to bring home the fact that the figures given do not represent a safe yield.

Richard H. Gould,* M. Am. Soc. C. E.—The paper† by Allen Hazen, M. Am. Soc. C. E., marked a distinct advance in the science of hydraulic calculations utilizing natural stream flow. The paper by Mr. Sudler represents an exhaustive study, utilizing many of the data and the method of analysis brought out in the earlier paper. It may really be considered an extension of the work started by Mr. Hazen. A study of this nature entails a tremendous amount of work and careful thought, usually much more than most engineers have an opportunity to spend for this purpose. The profession is, therefore, greatly indebted to Mr. Sudler for his results.

Because of the importance of the earlier work, it is gratifying to note that Mr. Sudler's independent study has shown that Mr. Hazen's seasonal storage curves can be depended on "in literally hundreds of instances". Mr. Sudler has suggested certain modifications in the higher rates of draft that are of interest. The speaker will confine his remarks principally to the seasonal storage curve for the lower rates of draft as applied to certain low stream flows.

A particular case in question is in connection with the yield of the impounding reservoir of the Cumberland, Md., water supply. These works, designed by James H. Fuertes, M. Am. Soc. C. E., were placed in operation

* With James H. Fuertes, Cons. Engr., New York, N. Y.

† "Storage to be Provided in Impounding Reservoirs for Municipal Water Supply," *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

in September, 1913. The collection works consist of a masonry dam about 70 ft. high, built across Evitts Creek, about 9 miles up stream from Cumberland. The reservoir formed is in the State of Pennsylvania, just above the Maryland State line. Water from this impounding reservoir, known as Lake Gordon, is filtered through a mechanical filter at the dam and flows by gravity in a 36-in. conduit to a distribution reservoir in Cumberland.

The water-shed area at the dam is 66 sq. miles, being about 18 miles long by 3 to 4 miles wide. The sides of the valley are steep, rising from 1 000 to 1 400 ft. above the creek bed, which is about 950 ft. above sea level at the site of the reservoir. The slopes of the hills are fairly well covered with timber, mostly of second growth. The soil is not deep, although there seems to be a considerable amount of fine sandstone on the water-shed. The reservoir has a water surface of about 5 900 000 sq. ft. when full and furnishes a storage, down to the lowest point at which gravity flow can be sustained, of about 1 000 000 000 gal.

A period of extreme dryness was experienced from June 17 to December 27, 1922, during which the reservoir was depleted to the extent of 810 000 000 gal. while sustaining an average draft of 7 000 000 gal. per day. The tax on the storage works was appreciably greater than had been anticipated in the design, and although it was possible to maintain a supply to the city at all times, the draft was reduced from a rate around 8 000 000 gal. daily at the start to about 4 200 000 gal. per day at the end. This was done without causing a great hardship, as Cumberland has a high water consumption per capita. Some anxiety was naturally felt as to the sufficiency of the works, which condition prompted further investigation as to the probable yield of the source.

The methods developed by Mr. Hazen seemed to furnish the best basis for this study, and if they are strictly applicable, indicate that the conditions of dryness along Evitts Creek in 1922 are among the most severe on record in the Eastern States. Based on the seasonal storage curves developed by Mr. Hazen and shown by Mr. Sudler (Fig. 6*), and including the evaporation from the reservoir as part of the draft, the period in question corresponds to a 99% dry year. In deducing this figure, no allowance was made for "ground storage".

The 1922 period on Evitts Creek is unusual, but it is by no means certain that it corresponds exactly to a 99% year. The normal storage diagrams developed by Mr. Hazen are based on curves that are inclusive of the actual records of Eastern streams, with one or two exceptions. The most notable exception in the storages required in a 95% dry year is that computed for the lower developments of the Tohickon Creek, near Philadelphia, Pa. These lie above the normal inclusive curve selected. The Evitts Creek data are similar to those from the Tohickon Creek in that relatively high storages are required in the small percentages of use. If the normal inclusive curve is raised to include the low developments of Tohickon Creek and the normal

* *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1925.

storage curves changed to correspond, the apparent degree of dryness on Evitts Creek would be appreciably less.

TABLE 4.—Flow of Evitts Creek at Lake Gordon, Cumberland, Md.

Month.	Total Monthly Stream Flow at Lake Gordon, in Million Gallons.									
	1914.	1915.	1916.	1917.	1918.	1919.	1920.	1921.	1922.	1923.
January	2 554	2 046	1 411	1 822	170	1 028	513	509	1 400	436
February	1 284	3 025	1 497	3 131	3 751	479	537	705	2 097	1 070
March	2 540	1 395	2 937	3 731	2 718	2 025	2 619	1 923	4 000	974
April	2 811	647	2 453	1 915	2 733	977	1 111	504	2 507	1 728
May	1 331	1 150	1 587	976	660	4 215	1 203	1 299	1 563	889
June	455	2 810	2 089	1 929	281	466	2 257	503	365	238
July	293	666	1 734	335	130	559	376	720	128	219
August	139	2 773	540	366	172	262	672	1 002	84	1 080
September	138	673	614	169	159	133	433	914	46	217
October	168	133	529	846	153	408	140	183	71	84
November	196	493	326	431	235	668	406	1 193	57	229
December	365	1 034	572	211	860	1 173	1 118	1 807	161	1 018
Totals	12 274	17 545	16 289	15 862	12 022	12 393	11 385	11 262	12 479	8 182

TABLE 5.—Annual Flows at Lake Gordon, Cumberland, Md.
(Co-efficient of variation, annual flow = 0.216.)

Year.	Draft, in million gallons per day.	Run-Off.		Rainfall, in inches.
		In million gallons per day.	In inches on 66 sq. miles.	
1914	9.6	33.6	10.7	32.3
1915	8.5	48.0	15.2	45.4
1916	8.0	44.6	14.2	35.5
1917	8.7	43.5	13.8	33.0
1918	10.3	33.0	10.5	34.0*
1919	9.6	33.9	10.8	34.3
1920	8.8	31.1	9.9	32.9
1921	8.0	30.9	9.8	37.0
1922	7.7	34.2	10.8	21.8
1923	8.5	22.4	7.1	36.8
Average	8.77	35.52	11.28	34.30

* One month assumed.

The experiences which were had with the Evitts Creek development are not sufficient evidence to modify the normal storage curve in its lower portions. It may be noted, however, that the evidence here is quite similar to that brought out in the important records of Tohickon Creek. These considerations should be kept in mind in arriving at conclusions as to the probable recurrence of periods of similar dryness. Because of the geographical location of Evitts Creek and the records of streams like Tohickon Creek, it is probably safer to assume that the 1922 dry period will recur oftener than once in a century. The value of the method of probabilities in classifying the 1922 dry period on Evitts Creek is clearly indicated.

The worth of the seasonal storage curve will increase from year to year as it is compared with additional records from different localities and found to be correct. The author has pointed out that the curve may require modification for use in some localities. Data that will establish the correctness or point out limitations in the use of the curve are of value. It is for this reason that the rather unusual records of the flows of Evitts Creek are submitted.

TABLE 6.—WEEKLY RECORD OF DEPLETION IN LAKE GORDON, CUMBERLAND, MD., IN 1922.

(Average draft, 7 020 000 gal. per day; storage used, 12 300 000 gal. per sq. mile.)

Period ending.	Use, in Million Gallons. Draft.	Evaporation from lake.	From storage.	Stream flow.	Remarks.
June 17, 1922 ..	....	....	....	....	Reservoir full.
" 24, " ..	54.8	5.7	15	45.5	....
July 1, " ..	55.6	6.8	10	52.4	....
" 8, " ..	55.8	5.0	25	35.8	....
" 15, " ..	57.5	6.0	20	43.5	....
" 22, " ..	56.9	6.4	30	33.3	....
" 29, " ..	55.4	4.9	45	15.3	....
August 5, " ..	59.1	4.5	10	53.6	....
" 12, " ..	58.3	4.2	40	22.5	....
" 19, " ..	61.2	5.1	45	21.3	....
" 26, " ..	49.0	3.7	43	9.7	....
September 2, " ..	66.0	3.6	42	27.6	....
" 9, " ..	57.5	5.1	40	22.6	....
" 16, " ..	66.6	4.0	50	20.6	....
" 23, " ..	57.7	2.6	59	1.1	....
" 30, " ..	54.6	2.0	56	0.6	....
October 7, " ..	47.5	2.5	50	0	....
" 14, " ..	38.8	1.5	20	19.8	....
" 21, " ..	36.0	0.9	30	6.7	....
" 28, " ..	40.9	0.7	20	21.6	....
November 4, " ..	43.2	0.8	30	14.0	....
" 11, " ..	39.8	0.5	20	20.3	....
" 18, " ..	38.8	0.6	30	9.4	....
" 25, " ..	44.2	0.3	20	24.5	....
December 2, " ..	42.3	0.4	25	17.7	....
" 9, " ..	39.2	0.4	8	31.6	....
" 16, " ..	34.3	0.2	12	22.5	....
" 23, " ..	29.5	0.6	5	25.1	....
" 27, " ..	17.5	0.4	10	7.9	Day of maximum depletion, 193 from start.
February 14, 1923 ..	....	....	....	....	Reservoir full.
Totals..........	1 356.8	79.4	810	626.2	....

Tables 4, 5, and 6 give the monthly and yearly records of the stream flow at Evitts Creek for a 10-year period, together with weekly records covering the period of maximum depletion of the reservoir in 1922. The records are subject to variation common to all hydraulic measurements of this nature, but are believed to be representative.

Mr. Arthur G. Fowler is the Superintendent in charge of the reservoir, filter plant, and pipe line, and it is to him that the speaker is indebted for the carefully kept records on which this interesting series of flows is based.

A. STREIFF,* M. AM. SOC. C. E. (by letter).†—The author states:‡ "The merits of the method of study here given [the Hazen method§] are generally unrecognized." The writer is unable to verify this statement, except by noting that the latest editions of the handbooks of hydrology do not give it in detail. In his practice, during the fourteen years that have elapsed since the first paper appeared, application on short-term records has shown certain inherent defects; but it must be said in all fairness that Mr. Hazen has warned against the use of these. Mr. Sudler, however, does not seem to concede these defects. While Mr. Hazen has expressed himself throughout with scientific caution, and merely states his belief in the greater accuracy of his method, Mr. Sudler takes the reliability thereof as a condition *sine qua non.*

This paper may be summarized as presenting a number of curves for rates of draft higher than those in the original paper, but it does not offer any additional proof nor investigate other rivers. Therefore, it cannot be said to have extended knowledge of the reliability of the method. Mr. Hazen's paper is one of the most important publications in the field of hydrology, but as time elapses and further studies are accumulating, certain inconsequences are becoming clearer. The findings of Mr. Hazen, far from representing probability methods applied to fortuitous numbers, might be classified as the establishment of certain simple laws of river flow based on statistics.

The writer, on the other hand, finds that these laws are not quite so simple, that wide departures therefrom may be expected. He does not admit that the computations given by Mr. Hazen, while unquestionably useful as rough approximations for just such problems as confronted Mr. Sudler, can be held to be more accurate than the ordinary method as used by the late Desmond Fitzgerald and by John R. Freeman, Past-Presidents, Am. Soc. C. E. In other words, Mr. Hazen, discovering certain simple relations in river flow, swings the pendulum too far in that direction.

The presentation of certain normal diagrams applicable on all rivers automatically dispels the notion that flow records are fortuitous numbers amenable to analysis by probability methods. As a matter of fact, little of the latter has really been used, and the whole procedure may be summed up in the following: The departures from the mean for a certain generally small number of years, n, plotted in the order of size are held to apply in the same proportion to any value of n.

In other words, the probability *a priori* is held to be equal to the probability *a posteriori.* This, in the theory of probabilities, is only true in case n is a very large number. If by plotting a comparatively small number of values in that manner certain normal diagrams result, this merely indicates the existence of certain laws applicable only to the comparatively short records used; but it permits no conclusion as to "95% years" and the like, covering an infinite future. Besides, within the range of time covered, the departures

* Cons. Engr., Bangor Hydro-Elec. Co., Jackson, Mich.

† Received by the Secretary, March 19, 1927.

‡ *Proceedings,* Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1918.

§ *Transactions,* Am. Soc. C. E., Vol. LXXVII, (1914), p. 1539.

of the individual rivers are quite large, and average values, applied on any one river, may be quite far from the truth.

The writer has advanced* a number of reasons for the belief that flow records are not unrelated numbers, but contain a large systematic element. This latter, on the other hand, is not as simple as expressed in Mr. Hazen's normal diagrams. It is not permissible to expect 300 or 1 000 flow-years to show the same proportions as a small flow record. In other words, flow records must be regarded as histograms, and, as customary in statistics, all trends must be thoroughly removed before the remainder can be subjected to probability methods.

About twenty-six years ago the late Elmer Corthell, Past-President, Am. Soc. C. E., while visiting the jetties at the Hook of Holland, expressed to the writer his belief that all rivers are subject to certain regular fluctuations not unlike the tidal flows in estuaries. As a matter of fact, scientific literature at that time already contained many data tending to show that artificial 1 000-year records are contrary to established knowledge. During such long terms of years considerable changes do take place, which have a tendency to recur periodically. Perusal of the extensive literature of climatic changes will soon convince any one that artificial long records can not be considered seriously. Reference to the studies of many scientific investigators dispels the possibility of adopting artificial fortuitous records. Such arrangements simply do not exist in Nature, and engineers can ill ignore the findings of the scientists.

Long flow records are not available, but long historical records of high and low stages of rivers and lakes exist, and these, reaching far back into history, have been carefully studied. The record of the Nilometer at Cairo, Egypt, kept during the period of Arabian occupation for eight centuries and given by Aboul Mahásin in the *Mémoires* de l'Institut d'Egypte, 1923,† shows plainly the periodic swings of long duration. The changes of the inland seas of Central Asia likewise indicate these long swings‡ which relegate 1 000-year artificial records into the realm of fancies.

Also, for shorter terms, the existence of periodic variations has been studied carefully. The famous book of Professor Brückner, Geographer at the University of Berne, Switzerland, published thirty-seven years ago, gives elaborate statistics proving the existence of three variable climatic cycles the shortest of which, now called the "Brückner cycle", has a period of ± 35 years, but varying from 25 to 50 years.

Considering the still shorter periods of time, there are many studies covering small cycles of 2.5 to 3.5 years by Helland Hansen, Nansen, Bigelow, Lockyer, Axel Wallén, Braak, Arctowski, Clough, and others. It is plain

* *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1841.

† "Ritmi cosmici nelle oscillazioni climatiche con speciale riferimento alla serie pluviometrica Patavina," G. del Valle, *Atti e Memorie* della R. Academia di Science, Lettere, ed Arti, in Padova, 1926, Vol. XLII.

‡ "Le niveau des lacs de l'Asie centrale russe et les changements de climat," J. de Schokalsky, *Annales de Géographie*, Vol. 18, pp. 407–415.

that fortuitous flow records can only be seriously considered after errors in the statistics of these investigators have been proved.

Neither can the amplitude of these systematic variations be neglected, especially for storage studies. The amplitude of the Brückner cycle in the San Diego, Calif., rainfall is more than 35% of the mean; the 11-year cycle was responsible for a 56% variation in the flow of the Huron River, in Michigan. These percentages are variable.

Such cycles cause the erratic fluctuations of some of Mr. Sudler's computations. For instance, he gives, for the coefficient of variation of the Croton River, for,

Ten 5-year means,	maximum =	827%	of minimum
Five 10-year means,	"	204%	" "
Two 20-year means,	"	122%	" "

There is no direct evidence to show how this coefficient would have varied if twenty 20-year periods had been available. It is plain that Mr. Hazen's deduction based on the formula:

$$\frac{\text{Probable error in one term}}{\sqrt{\text{Number of terms}}}$$

is only applicable to fortuitous records, and does not hold for histograms.

It is significant that Mr. Hazen fully recognized the importance of the occurrence of cycles, and admitted the necessity of critically examining longer precipitation records of the region involved. Mr. Sudler, however, seems to regard the method to be more firmly established, stating, regarding short records, that "only one simple calculation is needed which may be based on a short-term flow record"; and, again, "to apply these to a particular stream for which only a few years of flow record are available", etc. In a case of the Huron River, in Michigan, where, due to the 11-year cycle, the four years, 1913-16, have a mean flow 56% higher than the four years, 1920-23, the determination of the storage constant from such a short record is evidently entirely false. Mr. Hazen, on the other hand, has specifically warned against the errors of short records. Also, within the year itself the existence of the cycle is evident, and, again, of not nearly as simple a nature as that expressed by Mr. Hazen's normal storage diagram.

The normal storage diagram is an average, from which wide departures may be expected. As a matter of fact, Mr. Hazen has already admitted this by stating that it represents climate. Far from indicating a probability of fortuitous nature, the normal storage diagram is a simple geometrical consequence of the law of yearly variation in river flow. Drawing a mass curve of monthly flows, it may be shown that within the year the low point of the curve usually occurs at a certain time, six to nine months later than the highest point. It is plain that when such a time difference between the two points is constant, the increments in storage for equal increments in draft will be equal.

This is a simple fact of plane geometry, entirely independent of any run-off characteristic except the time difference between the high and low points

of the mass curve within the year. Likewise, the increasing dryness of the years tends to lengthen this distance, which, for each river, is quite a variable quantity from year to year. For rivers with widely different time differences between high and low points of the mass curves (such as found in the Pacific Northwest with its winter rains, where this time difference is apt to be three months), the normal storage diagram diverges still more; but for the individual rivers investigated by Mr. Hazen the differences are also considerable, and he arrives, by a process of smoothing and averaging, at a normal diagram that may be accepted as an approximate average, but hardly can be considered "more accurate than any but the longest flow records" of an individual stream. The normal storage diagram does not even apply to all rivers on the Atlantic Coast; the Penobscot River, for instance, requires a much greater seasonal storage.

Not only is it impossible to determine the storage constant from a short record with any measure of reliability, but if a reasonably long record is at hand it will take as much time to determine the constant as it does to determine storage. Furthermore, if, in agreement with scientific opinion, the view is held that the run-off record is a "historigram",* the normal storage diagram loses the value ascribed to it, for the storage constant may be quite different for another phase of the Brückner cycle.

The computation of yearly storage based on the coefficient of variation also pre-supposes simpler laws of river flow than really exist. It is to be regretted that for the departures from the mean the term, "variation", is used, for, in statistics, this term is also used to indicate the difference of consecutive terms, the departures from the mean being termed "deviation" (Goutereau ratio). To state the draft in terms of the deviation from the mean is a clever artifice to reduce all rivers to a relative basis; but the question is whether this purpose is really thereby attained. This fact is only proved by Mr. Hazen† by allowing a large degree of approximation.

Beyond the length of one year the amount of storage is again determined by the presence of cycles in the record. The depletion diagram is a mass curve, and thus the amplitude of the longer cycles is enhanced, and the shorter suppressed, in amplitude. The most prominent cycle present in the length of the records used is the 11-year cycle and this is, therefore, the largest single factor in determining the annual storage; but there are others. These, combined, render the annual storage curve as dependent on many causes as the seasonal storage curve. Again, proportionality of increments of draft and storage only occurs if the distance between high and low in the mass curve is constant, and this occurs in the annual storage curve in discontinuous fashion; the control switches from the smaller cycles to the 11-year cycle and from this to the Brückner cycle, just as with increased draft the control switches from the seasonal to the annual curve. This would become clear to any one who would plot the mass curve of the Tennessee River, for instance, at Chattanooga, Tenn., one of the longest records available in the United States.

* A graph which has time as abscissa and run-off as ordinate.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1595, Plate XXXVI.

Mr. Hazen then adopts a quasi-fortuitous arrangement by taking various records and placing these in succession regardless of their place in the scale of time. Mr. Sudler reduces the systematic features still more by the adoption of merely a certain degree of skew in selecting a certain duration curve, and building from it a storage diagram. The coefficient of variation does not completely describe the habits of the river, for $\sqrt{\frac{\Sigma \Delta^2}{n}}$ may be constant for entirely different groupings of Δ. This precludes reduction to the same relative basis by the coefficient of variation, for the storage required is, for the same coefficient, also dependent on the degree of skew.

Neither of the two methods appears to be admissible, as soon as one accepts the fact that the flow record is a histogram. Accepting a fortuity theory the method of Mr. Sudler would seem to be more consistent, as Mr. Hazen preserves the records, arbitrarily chosen as to length, intact, but joins them in fortuitous fashion in the scale of time. It seems that for most streams a short record exists sufficient—according to Mr. Sudler—to determine the coefficient of variation; but is it to be concluded that the expensive stream gaugings of the U. S. Geological Survey may as well be discontinued, as the values are considered entirely fortuitous anyway? A perplexing task confronting the supporters of the fortuity theory is to point out errors in the statistics and conclusions of Professor Brückner's work,* wherein the climatic cycle is derived which changed, for instance, the mean flow of the Tennessee River 30% in the period, 1882-92, over the mean flow of the next 12 years.

As the matter stands, Mr. Hazen, while fully recognizing the influence of the cycles, considers these of minor importance. This belief the writer does not share. One instance of a large error was pointed out by Mr. F. B. Marsh.† The writer might mention the Huron River, showing a difference of 5.6 times the probable error in two 4-year periods, four years apart; the Tennessee, showing 2.5 times the probable error in one 10-year period; and the Penobscot, 4.3 times the probable error between two consecutive 4-year periods. The averaging of probable errors is no criterion, for, after all, the statistics are applied to individual cases.

The determination of the total storage curves by Messrs. Hazen and Sudler seems to the writer wholly inadmissible. The addition of the annual to the seasonal or monthly storage—on a somewhat different basis by both authors—can not be admitted, as they are not simultaneous values. The maximum depletion in annual storage occurs not necessarily (in fact, seldom) in the year of minimum flow. Neither does the maximum seasonal storage, although as Mr. Sudler's Fig. 10‡ shows, it follows the degree of dryness in a general way. The two are not related, and may occur in entirely different years. It should not be overlooked that the depletion diagram is a mass curve, hence it lags 90° in phase behind the hydrograph, the dryest year not

* "Klimaschwankungen seit 1700".

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1644.

‡ *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1929.

occurring at the maximum depletion. Referring again to Mr. Hazen's Plate XXXVI,* the Croton River shows a maximum depletion for what might be estimated as a 70% dry year; taking the mean curve of Mr. Sudler's Fig. 10 the 70% seasonal and 95% annual should be added, and not as in Mr. Hazen's Table 15,† both 95% values. This is an important point which introduces errors in both Mr. Sudler's and Mr. Hazen's total storage curves, and which again illustrates the difficulty of attempting to eliminate the time as a dimension of the flow record.

Finally, if interpreting Mr. Sudler's Fig. 13‡ correctly, the writer finds very considerable differences in the amounts of annual storage between his and Mr. Hazen's diagrams:

Draft = 1 — 0.4 C,	80%	dry	year,	116%	difference in	storage
" "	90%	"	",	55%	"	"
" "	95%	"	",	40%	"	"
" "	98%	"	",	24%	"	"
" "	99%	"	",	24%	"	"

It is, of course, quite important whether a reservoir should be, say, 40% larger or smaller; such divergent results, if correctly arrived at by the writer, can be characterized as gross approximations only. Since Mr. Sudler uses Mr. Hazen's values for the seasonal storage, the diagrams of both are alike.

Mr. Hazen considers the use of a mass curve giving only the accumulated departures from the mean, a contribution of real importance to the methods of making storage calculations. It is indeed surprising that it has not been in universal use, for it was published long ago.§ Such mass curves show the succession of groups of dry and wet years very plainly and are well suited to demonstrate the periodic structure of river flow.

The writer has freely criticized the paper and the method on which it rests, believing the subject of great importance and deserving careful consideration. He is not prepared to follow Mr. Sudler and Mr. Hazen in their attempts at such a sweeping simplification of the tangled skein of terrestrial phenomena as a normal storage diagram for all rivers and all time expresses; but he pays tribute to the great amount of thought and labor involved in these investigations, which undoubtedly have delineated many questions on river flow heretofore not discussed.

Sigurd Eliassen,‖ Assoc. M. Am. Soc. C. E. (by letter).¶—One of the notable ideas in this paper on the extension of Mr. Hazen's method for analyz-

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1595.

† *Loc. cit.*, p. 1597.

‡ *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1930.

§ *Water Supply Paper No. 279*, U. S. Geological Survey, Pl. X, p. 197, published in 1914, is just such a mass curve. The authors of this paper are H. K. Barrows and C. C. Babb, Members, Am. Soc. C. E. The writer has used such mass curves for many years, and published such curves in the *Monthly Weather Review*, July, 1926. *Bulletin No. 5*, Dept. of Public Works, State of California, an Appendix to the 1923 Report on the Water Resources of California, contains a great many mass curves of California streams in that form.

‖ Engr.-in-Charge, Survey Dept., Chihli River Comm., Tientsin, China.

¶ Received by the Secretary, March 28, 1927.

ing stream flow in connection with storage requirements is the author's manner of supplementing the data by means of chance drawings.

The statistician, with records at his disposal, such as would make an hydraulic engineer's mouth water, sneers at the engineer's attempt, in extrapolation from short-term records, to use the method of probabilities as a tool for guiding his judgment. "Keep within the limits of your data", says the statistician; but, unfortunately, that is just what the engineer cannot do. He must try as well as possible to argue beyond. Duration curves, Mr. Hazen's probability paper,* and, of late, Mr. Goodrich's skew probability paper,† are all devices for making the extrapolation safer. To this is now added the author's idea of "creating" an hydrological universe from a duration curve constructed from short-term records, and of making chance drawings from this universe.

The idea has considerable merit but the writer questions the author's method of drawing. It is not clear why the drawn card was left out after each drawing. Surely the tendency by not putting the card back would be sooner or later to force an unduly prolonged dry or wet period. Why not put the card back again, thus drawing from a universe of constant proportions? Even this is perhaps not strictly correct, as periodic changes in the solar radiation influence the conditions. However, as long as the universe is made up from a duration curve which in itself somewhat reflects the effect of these periods, no very serious error should result on this account as long as the water-shed is small. For large water-sheds the tendency to well marked periodicity in run-off may become an important factor.

What reliance can generally be put on a short-term record? Avoiding dubious mathematics the writer has taken a duration curve of maximum flood discharges of the Yung Ting River in China for the 15-year period, 1912 to 1926. Assuming it to reflect within a small margin of error the actual condition, he has made a universe of 100 records between the limits 6 000 cu. m. per sec. to 240 cu. m. per sec. Then 2 500 chance drawings were made keeping the conditions constant. The resulting "duration" curve is shown in Fig. 37 (*a*). It lies very close to the original curve which, of course, was to be expected. On Fig. 37 (*a*) are also drawn "duration" curves for the lowest and the highest 25-year period (25-year periods were used in order not to obtain absurd results). Two other chance series were also made from the same universe, namely, by choosing, at random, five 25-year periods and one 125-year period and dividing the latter into five consecutive 25-year periods. The "duration" curves were constructed for these periods and are shown in Fig. 37 (*b*) and Fig. 37 (*c*).

The object in constructing these curves is to show a few of the endless variety of universes it may be possible to obtain by slicing out short periods from a long-term universe. At first glance, even a 25-year record does not seem to offer much of a basis for building up a universe that will represent the actual conditions. However, by further considering the case it is per-

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

† *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1063.

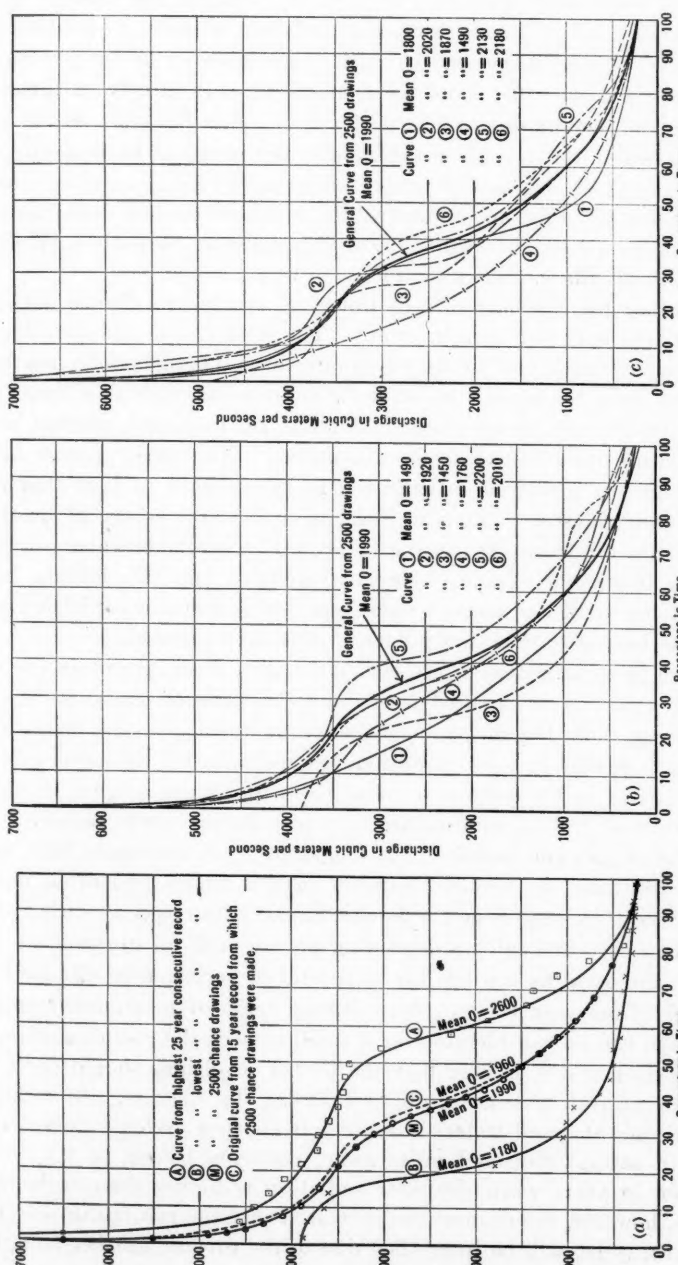

FIG. 37.—DURATION CURVES OF MAXIMUM FLOOD DISCHARGES, YUNG TING RIVER, IN CHINA, 1912–1926.

haps not as bad as it looks. It is evident that the chances are very small indeed that one should just happen to strike some of the wettest or dryest 25-year periods, which a very long-term universe is likely to produce. They would surely be noticeable periods much commented on by the population of the district. On the other hand the chances are very largely in favor of the period from which the universe is made up being rather close to the average condition, and if the records are used for the purposes of determining the longest sequence of dry or wet years a short-term record, in most cases, will serve well for building up a universe from which chance drawings can be made as suggested by the author. For extrapolation of such happenings as maximum floods the method may not be so well adapted.

The author pre-supposes that hydrological events are chance happenings, but this is true only to a certain extent. The total visible and invisible water content on the earth and in its enveloping atmosphere may be regarded as constant at least for periods of several hundreds of years at a time. Small variations of a returning character take place; or, putting it another way, the evaporation-precipitation cycle is fluctuating. The main factor bringing about changes in this cycle is variation in the amount of heat that reaches the earth from the sun. The other factors, such as the escape of small quantities of water vapor into space, the variations in available water content due to changes in vegetation and to human activities, and also leakage into the interior of the earth and escapes therefrom due to volcanic activities, may be regarded as having so slight an influence as to be negligible.

Changes in solar radiation are correlated with outbreaks of faculæ on the sun, which again are correlated with the appearance of sunspots. Based on accurate solar radiation measurement some meteorologists are now ready to assert that increases in solar radiation are accompanied by increased atmospheric activities on the earth and are directly the causes of anti-cyclonic and cyclonic storms.* The atmospheric pressure during such periods becomes lower in the tropics and higher in the temperate zones and again lower around the poles, resulting in increased rainfall in the tropics and often increased aridity in the temperate zones, although in the latter region rainfall may be intensified in certain localities depending on the paths of cyclones.

Taking the earth as a whole the sum total of variation in climate is relatively slight from year to year, even during years of maximum faculæ; yet the variation can be considerable for a small part of the earth's surface, such as a water-shed, and it is here that the matter of chance should enter. The tendency of rainfall to concentrate erratically; the striking, non-striking, or partial striking of small water-sheds by rain-bearing cyclonic "lows" having their origin several thousand miles away, certainly belong to the realm of chance even in years when the solar radiation is higher than ordinary. In such years, however, the chances are increased that the rainfall or the aridity, as the case may be, will be intensified due to the greater number and greater intensity of "lows" and "highs."

* "The World Weather," by H. Clayton.

A serious outbreak of sun faculæ usually lasts for a few months. Its effect on the atmospheric activity on the earth will be felt about every 14 days, or when the faculæ due to the sun's rotation appear on the side of the sun facing the earth, the sun's rotation having a period of about 27 days. It becomes a matter of chance, then, if for a particular area the outbreaks will coincide with the wettest or dryest period, or produce negative results.

Also, regions lying in the paths of both cyclonic "lows" and "highs" are generally more exposed to variations in climatic conditions than regions lying away from the general paths of these storms. North China, for instance, lies directly in the paths of the Siberian anti-cyclones and cyclones. These are most prevalent during the winter and early spring. During July and August the Pacific typhoons often cross it. Hence, aridity during the winter and spring and intense rainfall or drought during the summer are the prevalent conditions, and maximum faculæ periods seem to coincide either with the maximum droughts or with maximum flood periods.

If some one having solar radiation data at his disposal would examine the possibility of seasonal periodicity in connection with the 3.6 and 11.3-year periodicity, he could perhaps find an approximate law underlying variations in rainfall and run-off even for small water-sheds. At present, it seems that effects of cyclic changes in solar radiation on stream flow can only be shown for very large water-sheds, such as the Mississippi River and the Great Lakes. The laws governing the flow of smaller rivers are very obscure, but can perhaps be discovered when the basic facts governing the "disturbing" or, rather, the "order-producing", influences are unearthed.

Variations in solar radiation are now being intensively studied by means of bolometer measurements, the latest observatory being established in South Africa by the Smithsonian Institute.

RECENT DEVELOPMENTS IN CONCRETE PAVEMENTS

Discussion*

By Messrs. H. C. Boyden, C. Raymond Hulsart, and Edward Godfrey.

H. C. Boyden,† M. Am. Soc. C. E.—For many years there has been much discussion regarding the need for reinforcement in rural concrete pavements. Recent observations indicate that it pays from all points of view to use reinforcement of 60 lb., or more, per 100 sq. ft., and that the best results are obtained when the steel is in small, rather than large, units. This observation favors wire mesh at the expense of bars, but it also implies that the mesh shall have been properly placed.

Many engineers have objected to mesh because of the difficulties attendant on placing it properly in the pavement. To-day, however, with mesh shipped in flat sheets rather than in rolls, or with proper straighteners on the job when the material is shipped rolled, there should be no greater difficulties than when bars are used.

Reinforcement does not add materially to the structural strength of the slab; but it does prevent some cracks, holds together such cracks as do occur, and distributes impact from moving loads over greater areas.

C. Raymond Hulsart,‡ Assoc. M. Am. Soc. C. E.—The importance of early strength in concrete warrants further details than the author has given.

Certain field conditions sometimes make it difficult to secure a high early strength with standard Portland cement. For instance, the small quantity of mixing water, 4.4 gal. per bag of cement, will be difficult to use with many commercial aggregates. Also, the long-time mixing may be difficult or expensive to secure, in that it would cut the contractor's mixer output to about one-third of the normal. In using calcium chloride, mixed with the concrete, the engineer should have in mind that it re-acts differently with different brands of Portland cement and does not re-act at all with some brands. Tests with the brand of cement to be used will determine the exact quantity of calcium chloride required for the desired effect. Further, the proportioning and mixing with the calcium chloride is an accurate process, requiring careful attention.

This difficulty has encouraged the development of special quick-hardening cements, or early strength cements, especially in Europe. There, the so-called "super-Portlands" or "super-cements", which have quick-hardening properties,

* Discussion of the paper by H. Eltinge Breed, M. Am. Soc. C. E., continued from April, 1927, *Proceedings*.

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‡ Vice-Pres., The Atlas Lumnite Cement Co., New York, N. Y.

are generally secured by finer grinding and a high lime content. Even in Europe considerable difference of opinion exists as to their permanence and durability. These cements have not, as yet, been manufactured in America.

The quick-hardening cement longest in use in Europe (since 1912, principally in France) is not a Portland cement, but an alumina cement made from aluminum ore (bauxite). The product is known as "Ciment Fondu". Previous to the World War, its use was principally on account of its resistance to the chemical attack of sea water. During the war it was used for the construction of heavy artillery foundations which could then be put in service in 24 hours. This gave aluminate cements an impetus which has brought about their rather general use throughout Europe.

By high early strength is meant that within 24 hours they become considerably stronger than other cements in 28 days. Aluminate cement known as lumnite has been used in America since the summer of 1924 to meet special conditions, such as intersections of roads and streets, or for certain short pieces of road where traffic could not long be interrupted. For example, State highway departments have used aluminate cement for important intersections that have been poured one day and thrown open to traffic the next. Many highway departments have used aluminate cement also for the repair of concrete pavements and for replacements.

By way of illustration, in Maryland, a road was to be constructed leading to a pass under a railway. There were banks on each side; the traffic could not be detoured conveniently, and it was so heavy that it could not be stopped for any considerable length of time. The State Highway Department constructed a section of pavement with aluminate cement, finishing it on one afternoon and on the next throwing it open to heavy motor traffic.

Likewise, in New York City, at Lafayette and Houston Streets, according to the Chief Engineer of the Borough on the heaviest continuous traffic artery in the world, a section of the street was closed on Friday night, the concrete base was placed with aluminate cement, the granite block put on top of it, and the road was thrown open to traffic on Monday morning.

These aluminate cements, due to their rapid chemical action in hardening, develop considerable heat, so much so that they have been found safe for use in cold weather. For example, near Netcong, N. J., there was 1 200 ft. of road under construction and unpaved in December, 1924. It was not considered desirable to leave it in that state through the winter. This stretch was finished on Saturday night in freezing weather, using aluminate cement, and was thrown open to traffic on Monday morning.

Edward Godfrey,* M. Am. Soc. C. E. (by letter).†—Mr. Breed's paper offers evidence of the fact that the problem of the best way to lay concrete pavements has not been determined or at least agreed upon. The matter of shrinkage cracks and their avoidance has always been a problem difficult of solution. One of the standard methods of building concrete sidewalks is instructive on this point. A perfect bond between the base and the finish

* Structural Engr., Pittsburgh, Pa.

† Received by the Secretary, January 24, 1927.

coat may be obtained by using mixtures having mortar of the same richness for each layer, and by placing the upper course before the lower has set. This avoids any possible unequal shrinkage such as often takes place in the case of mortars of different richness. This danger is also an argument against two-course construction in road work unless the second course is placed immediately after the first one is poured.

Instead of being laid in long lengths at one time, the pavement might well be blocked off into sections and alternate slabs poured first, followed by the intermediate sections. In this way one-half the shrinkage of setting is eliminated entirely, and the shrinkage cracks, if any occur, are definitely located at the construction joints.

In the ordinary way, where long lengths of road paving are placed without break, except by the introduction of separators or by the questionable method of driving in a plate to cut up the concrete, it cannot be expected that the best results will be obtained. When one or more squares or oblong blocks of concrete are poured against bulkheads and allowed to set, all the shrinkage of those blocks is taken up. Then when the bulkheads are removed and the intermediate blocks are poured, two things are accomplished: First, the longitudinal shrinkage of setting is only that of the second set of blocks; and, second, there is little adhesion between blocks, although a perfect fit, and any tendency toward shrinkage cracks is forced to these construction joints.

Mr. Breed makes the statement* that, according to tests made by Professor Abrams, wetter concrete, among other things, means more permeable concrete. He intimates that compressive strength and impermeability vary with diminished quantity of water in the mix, and that the former quality is a measure of the latter. This is a very commonly held notion. It is, however, not shown by Abrams' tests to be true.

The tests made by Abrams† are on absorption and not on permeability. There is an essential difference. Permeability allows the passage of water through the mass, whereas absorption is a measure of the water that enters the pores of the mass. A cork will absorb water but will not let it pass through the mass. A sound cork will absorb more than 50% of its weight of water. A spongy mass of iron may be quite non-absorbent but may allow the passage of water freely. Much concrete will allow the passage of water freely, although it may not be particularly absorbent. The writer has seen concrete that had large spaces between the stones, but had the appearance of a non-absorbent concrete. The type of dry, tamped concrete that was universally used nearly thirty years ago was famous for being able to allow water to flow through thicknesses as great as 30 ft. It is to be noted that this is the zone where Abrams finds the greatest strength.‡ It is certain that this concrete would be neither non-absorbent nor impermeable.

* *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 36.

† *Bulletin No. 2*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

‡ *Loc. cit.*, p. 6.

Abrams' tests on absorption are few in number for a determination of so great importance, especially when compared with the hundreds of thousands of tests on the less important property of compressive strength. For reinforced concrete work a property of concrete of greater importance than compressive strength is impermeability, for the permanence of reinforced concrete would be jeopardized, if the concrete itself were not impermeable and did not protect the steel.

As stated, Abrams' absorption tests do not cover the zone of high compressive strength, as the water ratio extends only to 0.66 as a lower limit, and the strength curve takes a sharp upward turn below this ratio. Besides, while the specimens not kept wet during curing show a strong upward trend in absorption in the wetter mixes, the specimens stored in damp sand do not show similar results. Furthermore, there are comparatively many examples where the specimens cured in damp sand show much less absorption in the wet mixtures than in the dryer ones. The specimens cured in damp sand are really the properly treated ones, and their showing is of much greater value than the others. These indicate a trend toward greater absorption both in the wetter and the dryer mixes and a minimum absorption at water ratios between 0.73 and 0.91, which is the zone of average strength and consistency (and includes freely flowing concrete) and not that of high strength.

CONSTRUCTION METHODS ON THE MOFFAT TUNNEL

Discussion*

By Messrs. T. Kennard Thomson and H. F. Dunham.

T. Kennard Thomson,† M. Am. Soc. C. E.—The two things that make engineering really worth while are the contacts with fine men and interesting problems. This paper brings up recollections of forty-four years ago, and it would be a pleasure to tell about many of the then young men who have since made international reputations. However, only one who has passed on will be mentioned, the late Albert Brainerd Rogers, M. Am. Soc. C. E., the only engineer to discover two passes through the Rocky Mountains, one of which was mentioned by the author. Hundreds of interesting stories are told about this engineer, who discovered "Rogers Pass" (now called "Connaught"), which gave a short cut of 67 miles instead of 163 miles around the Columbia River. The discovery resulted in the Canadian Pacific Railroad awarding to Major Rogers a bonus of $5 000. Later, Mr. William C. Van Horne asked him why the check had not been cashed and was told that never before, in nearly sixty years, had any one paid him a cent more than obliged to—so he had framed the check. Mr. Van Horne again sent for Major Rogers and showed him a gold watch which was to be given him when the check was cashed. This overcame his reluctance.

On the Canadian Pacific Railroad through the Rockies some of the tunnel problems were similar to those at the Moffat Tunnel, but with very different working conditions. The timber lining and sections were similar. One of the earlier tunnels was distorted several times and finally collapsed, due to shifting clay, although the timber lining before the final collapse was a solid 12 by 14-in. wall. It was then abandoned and a temporary line (requiring a 23° curve) was used, until years afterward, a longer tunnel was built in a much safer location.

The earlier tunnel work suffered from the lack of high-grade tools, electric current, and even coal, all supplies except wood and water coming through Winnipeg, a thousand miles away, much of it from Chicago and Ontario. Nor were there any blueprints, ready made India ink, or typewriters. The labor problems likewise were radically different as all labor had to come from the East, and the men could not quit, although the payrolls were sometimes very late in arriving. Even bathing was difficult.

* This discussion (of the paper by R. H. Keays, M. Am. Soc. C. E., published in February, 1927, *Proceedings*, and presented at the meeting of March 2, 1927), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York, N. Y.

Only those who know the Rockies can realize how fortunate it was that, in crossing the Continental Divide, the Canadian Pacific Railroad did not have to reach any such elevations as the 11 660 ft. of the Denver and Rio Grande Western Railroad in Colorado, nor even the 9 200 ft. of the Moffat Tunnel. The highest point of the Canadian Pacific base of rail is 5 300 ft. at the summit, in British Columbia, not very far west of Banff. Elevations such as those in Colorado would probably have been prohibitive in British Columbia. The Canadian National Railway, much farther north, is still more fortunate with a maximum elevation of 3 717 ft.—a kind dispensation of Nature.

The author mentions exploders. In the East River (New York City) it was necessary to do some submarine blasting by drilling a number of holes under the bed of the river from a coffer-dam, and exploding all the charges at one operation. The makers of the exploders sent an expert to supervise the loading and wiring, but, unfortunately, only a few of the charges were detonated. On connecting about twenty-five similar exploders in the same manner, but without the dynamite, and sending the current through them, only four or five went off. Several efforts were necessary to get all the exploders detonated, which illustrates the improvements that have been made in explosives.

It is interesting to note that the same kind of gneiss rock as extends under New York City was encountered in the Moffat Tunnel and that it was decomposed 100 ft. or more below the surface. In New York, so far, this "rotten" rock has been found only near the surface, although as much as 50 ft. (in depth) of this soft rock has been removed.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).†—In the Ohio Valley District there is a tradition—possibly unwarranted—that oak timbers as large as 12 by 12 in. decay far more rapidly than timbers 6 by 12 in., spaced ½ in. apart for air circulation, where "heavy ground" requires a continuous 12-in. oak lining.

Every tunnel engineer knows that the integrity of his work depends largely on maintaining the exact position of the wall-plates. Years ago when in charge of such heavy ground construction for double-track steam railway in West Virginia, the writer found that carefully placed wall-plates were continually being disturbed in removing the bench. The contractor was ordered to secure and use 60-ton jacks in the re-alignment of the wall-plates. After the first few days the hydraulic jacks acted by catalysis only.

The instrument work for alignment and grade in long tunnels offers a chance for improvement over earlier conditions by using electricity. If one "bright spot" of known position could be shown when desired, on the face of a heading, or maintained all the time, the use of a light template would ensure a correct outline for the whole heading, tending to save needless rock removal and replacement.

The line of probable least disturbance in the average tunnel would probably be near the wall-plate line, say, 1 ft. inside and 1 ft. above that line. In

* Civ. and Hydr. Engr., New York, N. Y.

† Received by the Secretary, March 15, 1927.

most rock tunnels there is firm rock along the side walls within distances of 500 or 1 000 ft. At such points a vertical piece of timber could be anchor-bolted to the wall holding a bracket strong enough to support a wye-level on a base similar to that used by an instrument maker in adjustments between artificial, infinite-distant points. The level supported on such a bracket and brought into line by means of proper back-sights should then be reversed in its wyes, thus carrying both grade and alignment with great accuracy to the heading. A small platform for the operator would be required and a fender guard might possibly be needed, but there should be several hundred feet between the last bracket and the heading.

The next step should be to require of the instrument maker a light cylinder of the exact size of the level barrel, to be equipped with a light bulb and with suitable lenses to converge the rays and take the place of the level barrel in projecting the desired bright spot. A simple correction of 2 in. for every ½ mile would provide for earth curvature.

ECONOMIC AND ENGINEERING PROBLEMS OF HIGHWAY LOCATION

Discussion*

By Messrs. H. C. Boyden and John P. Hallihan

H. C. Boyden,† M. Am. Soc. C. E.—Referring to the real solution of many of the problems of highway traffic, the statement has been made that "to enter into an economic study of the location of highways, we have to enter into the realm of prophesy".

The engineer who is the best prophet is also the greatest dreamer and one cannot drive over American highways without realizing that they have been laid out by realists and not by dreamers or prophets. Most highways have been located with regard to the conditions existing at the time, without any serious attempt to provide for future traffic.

Who can prophesy what the volume of traffic will be fifteen years from now, any more than the traffic of to-day fifteen years ago could have been prophesied?

Only a very few years ago the (so-called) saturation point for automobiles was set at 17 000 000, and there are more than 20 000 000 already (not quite 1 per family) with no signs of the end. Now the automobile manufacturers, who are dreamers as well as hard-headed business men, are planning their future and spending their money on a basis of 2 cars per family. Are the engineers of to-day dreaming the same dreams and building accordingly? Quite evidently not.

A college professor, in discussing this subject, stated that, "before this dream comes true we will be traveling and carrying freight by airplanes". Undoubtedly, this dream will also come true, but history shows that a new form of transportation never lessens the use of any economical means of transportation used previously. The building of elevated railroads did not lessen the travel on the surface lines, nor did the building of subways put the elevated and surface lines out of business, as was prophesied.

Fifteen years ago, when California's system of State highways was started, no one dared dream that the car mileage on the roads of the State would be nearly 15 000 000 000 miles in 1927. Who dared dream that the time would come when the people of Illinois could spend $300 000 000 for a paved highway system of 9 000 miles, could pay the interest charges, the sinking fund, and the maintenance costs, and, at the same time, effect an annual saving of more than $80 000 000?

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* Discussion of the paper by W. W. Crosby, M. Am. Soc. C. E., continued from April, 1927, *Proceedings*.

† Lecturer, Celite Products Co., Los Angeles, Calif.

In locating highways two factors should be used as guides, namely, speed and safety. The end of "maximum speed" laws is in sight, because such laws are both economically and psychologically incorrect. Present and future highways must be made to carry more traffic than they are carrying to-day and this can never be done with the existing speed limits.

The killing and maiming of the hundreds of thousands of Americans annually by automobiles is not because of excessive speed in driving, but because the highways are seldom built to carry the volume of traffic they must carry to-day. How much less are they built to carry the traffic of one's dreams, when these dreams come true?

JOHN P. HALLIHAN,* M. AM. SOC. C. E. (by letter).†—The author has brought out very forcefully the economic and engineering problems in highway locations, which have been ignored to a considerable extent in many of the roads that have been built in the last ten years. It is only within that time that the question of volume of traffic on main highways has become of very great importance, except in the immediate environs of large cities. Now, with the whole nation "on wheels," and with the automobile habit increasing by leaps and bounds, the question of volume must be given due consideration, and the "ox-cart" and "man-and-horse" yardsticks formerly used for the width of roads must be finally discarded.

The cost of a 20-ft. pavement is about $30 000 per mile on the average, and, generally speaking, in the National highway system, such a pavement is laid on a right of way of 66 ft. If the road were always to remain a country road of light traffic the width of the right of way would perhaps not have an important bearing on the location, but the moment that a settlement of any kind grows up along this road the width of the right of way becomes insufficient, and this insufficiency becomes more marked as the settlement progresses from a village to a town and from a town to a city.

The writer is of the opinion that no main transportation artery in a city, or between two cities of importance, should have a right of way of less than 120 ft. Such a width, after deducting one-quarter of the right of way for sidewalk use, would leave 90 ft. for pavement, giving room for four lanes of travel and a parking lane in each direction.

It has been demonstrated that the 120-ft. right of way carries certain advantages. The traffic capacity per lane increases rapidly with the width of street up to four lanes, but with greater widths at a much slower rate. If, in a congested district in a large city, it is desired to separate the traffic on any thoroughfare from that of a cross-street, a width of right of way of 120 ft. permits the two center lanes, one in each direction, to be elevated or depressed, without material damage to the abutting property as would be the case if the grade separation were attempted in a street of lesser width.

In a large city demanding rapid transit facilities, a width of 120 ft. between building lines for main thoroughfares increases greatly in economic value. It permits the construction of four tracks underground in the most economical

* Chf. Engr., Rapid Transit Comm., Detroit, Mich.

† Received by the Secretary, March 16, 1927.

manner because of reduction of underpinning costs, and, at the same time, permits more convenient arrangement of loading platforms at stations.

Considering the fact that over any considerable mileage of road in a fairly well-settled region the average cost of right of way will not exceed, say, $500 per acre, the addition in each mile of 6.54 acres required to obtain a 120-ft. right of way instead of a 66-ft. width would amount to only $3 270. This amount would add about 11% to the capital investment of $30 000 in the 20-ft. pavement. Under such circumstances no economist would question the wisdom of protecting the future usefulness of the investment by obtaining the additional right of way before laying the first pavement, nor of making that provision for the future a part of the cost of the original construction.

Mr. Crosby also draws attention to the fact that the volume of traffic to be carried warrants fitting highways more closely to the topography. Certainly in view of the probable increase in ownership of automobiles in the United States to one for every three persons, within the life of any road now built, the expenditure of considerable sums to eliminate sharp curves and steep grades on important highways would seem to be justified, as well as the departure in location of the most important arteries from the strict adherence to the section lines, as is now generally the case. On main highway routes it would be well to consider the desirability of by-passing the cities along the route at a convenient distance, near enough that they would be readily accessible, but far enough away that the through traffic would not be compelled to set up interference with the traffic of the city itself.

The warning sounded by Mr. Crosby that the location of any highway of the present should be considered more carefully with relation to the future volume of traffic and to the lessening of interference on main highways with traffic of subordinate roads and of cross-roads, is very timely, particularly with respect to the highway development in the environs of large and growing cities.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

HARVEY JACOB DRESSLER, M. Am. Soc. C. E.*

DIED DECEMBER 9, 1926.

Harvey Jacob Dressler was born at Dayton, Ohio, on December 31, 1886. His early education was received in the public grade schools and High School of Dayton. Later, he entered Ohio State University at Columbus, from which he was graduated in 1909, with the degree of Civil Engineer.

From 1910 to 1912, Mr. Dressler was employed by the City of Dayton as Assistant City Engineer. The partnership of Cellarius and Dressler, Civil and Consulting Engineers, was formed in June, 1913, by Frederick J. Cellarius, M. Am. Soc. C. E., and Mr. Dressler, with headquarters in Dayton.

During the World War Mr. Dressler served with the 308th Engineers in France from June, 1917, to July, 1918, and after the signing of the Armistice, was with the Army of Occupation at Coblenz, Germany, until early in 1919.

He died on December 9, 1926, at St. Elizabeth's Hospital, in Dayton, while submitting to an operation for the removal of his tonsils.

Mr. Dressler is survived by his widow, Mrs. Florence Dressler, and two daughters, Mary Eleanor and Dorothy Ann.

He was a member of the Engineers Club of Dayton, the American Legion, St. John's Lodge of Masons, and the Scottish Rite. He was also an active member of Hope Evangelical Lutheran Church, in Dayton, where he led a Young Men's Bible Class.

Mr. Dressler was elected a Member of the American Society of Civil Engineers on March 15, 1926.

ALLEN DOUGLAS DUCK, M. Am. Soc. C. E.†

DIED JUNE 3, 1926.

Allen Douglas Duck, the son of Joseph George and Minnie (Fink) Duck, was born at Beaver Falls, Pa., on August 2, 1884. In his infancy the family moved into Ohio, locating, for a short period, at Salem. They then removed to Wellsville, in which town Mr. Duck received his public school education.

On graduation from High School he secured a place with the City Engineer of Wellsville as a Rodman on work being done there. Rapidly advancing from this position, he became Assistant City Engineer, which place he ably filled until late in 1904.

* Memoir prepared by C. S. Bennett, Assoc. M. Am. Soc. C. E.

† Memoir prepared by Malcolm Ramsay, Assoc. M. Am. Soc. C. E.

In January, 1905, he entered the service of the United States War Department, where, in teaching mathematics, in surveying, in laying of submarine mines and cables, in handling high explosives, etc., he laid the foundation on which later was built his enviable record as an engineer. After almost six years of this service he resigned his position with the War Department to take up general engineering practice with Prout and Wells, of Aurora, Ill.

As he was strong of body, and a keen lover of out-of-door life at its best, the opportunity for satisfying such a nature seemed to lie in Government service; so he returned to that service as a Recorder on Topography with the United States Geological Survey. After covering many of the States in this work—making an enviable record as an Assistant Topographer—Mr. Duck went to Greenville, Tex., in December, 1912, to accept the position of Assistant City Engineer. His valuable experience, together with his ability and capacity for hard work, won for him after a short time the place of City Engineer.

Devoted to his profession, and with the welfare of his adopted home grounded deeply within him, he began a task of construction and re-construction which grew with the years. No improvement of the city was contemplated without being first submitted to him for approval. To look after the need of a fast-growing city called for just such a man as Mr. Duck. Untiring, happy in the accomplishment of some difficult task, he spent whole days and nights in the solution of his more difficult problems.

Completing an extensive improvement program in Greenville in 1918, he tendered his resignation as City Engineer and entered private practice, acting as City Engineer in several Texas municipalities in the design of sewage and water-works systems, or the paving of their streets.

In 1919, he was called on by the County Commissioners of Hunt County, Texas, of which Greenville is the county seat, to accept the position of County Engineer to supervise the design of seven cardinal roads running through the County. He accepted this appointment and carried the work through until the project was almost completed, when with reluctance the Commissioners accepted his resignation, and he again devoted his time to the municipal problems of Greenville.

The constant drain on his facilities at this time undermined his health and he was compelled to stop work and seek rest for a short period. His indomitable will soon seemed to gain the upper hand in the struggle, and he again plunged into his work, seemingly as robust as ever.

In 1925, Mr. Duck joined the ranks of contractors, as a Contracting Engineer, accepting a contract under the City of Greenville to construct all concrete features of its new reservoir. Completing this work, he accepted, in the spring of 1926, a contract to pipe this water to the city in large mains. All construction machinery had been purchased and was on the ground, all material had been ordered, and everything made ready to commence the work when Mr. Duck was suddenly stricken at his home and passed away before medical aid could reach him. Thorough in all under-

takings he was one who believed that once a task is taken up, it should be worked through to a conclusion, and, knowing this, his wife stepped into his place and brought the work to a successful conclusion.

To have known Mr. Duck was a great pleasure—his personality, like the rays of the sun, lighted the dark places and warmed the cold. To have worked beside him was an honor. Ever alert to his task, he was looked on by his business associates as the personification of honesty and integrity. His father died during Mr. Duck's infancy, necessitating the curtailment of his technical education. He had the will to do; hence, by home study and with a clear mind as an asset, he surmounted this obstacle to his success. At the end of a far too short life, he was abreast of the foremost engineers of Texas.

A self-made man, Mr. Duck generously assisted those young people who were forced, through like circumstances, to travel the road over which he had so recently come. Generously, and ever quietly, he helped to smooth life's pathway for many of them. The esteem in which he was held by his fellow townsmen was manifested during his funeral services, when all municipal activities ceased for several minutes as a mark of respect to the man who had meant so much in the life of his town.

He was united in marriage on January 27, 1914, to Mildred McSpadden, of Greenville. To this union were born two children, Douglas and Mildred Frances. His home life was ideal, and his whole life was centered on those who formed the little family circle.

Mr. Duck was a true Christian and a member of the Christian Church. He belonged to all branches of the Masonic fraternity.

During the last few years of his life he became interested in stamps and had acquired a wonderful collection. He was a member of the American Philatelic Society. His community will long remember him for his engineering ability—his friends for his manhood.

Mr. Duck was elected an Associate Member of the American Society of Civil Engineers on September 12, 1916, and a Member on October 8, 1918.

MARSHALL MORRIS, M. Am. Soc. C. E.*

Died September 29, 1925.

Marshall Morris was born in Louisville, Ky., on May 30, 1846. He was the youngest son of Joseph Saunders and Eliza (Morton) Morris and a grandson, on the maternal side, of John Hite and Sarah (Price) Morton, of Lexington, Ky. His father was the seventh generation of the Morris family in this country.

Thomas Morris, an ancestor, married Abigail Marshall, granddaughter of Christopher Marshall. The Morris and Marshall families were of Philadelphia, Pa., both families having come from England in early Colonial days,

* Memoir prepared by J. M. Johnson, M. Am. Soc. C. E.

the Marshalls to escape Quaker persecutions. Christopher Marshall, with a number of others, was excommunicated by his Church for assisting the Revolutionary cause. These men formed the Society of Free Quakers, in which the older men gave their time and means to aid the Revolution and the younger ones joined the Army.

In 1863, at the age of seventeen, Mr. Morris was graduated from the University of Louisville, receiving the degree of Bachelor of Science.

Immediately after graduation he joined a surveying party on railroad location work in Eastern Kentucky and was employed on that work for several years, being Chief of Party on the survey of the Louisville and Cincinnati Railroad in 1866.

In 1871 and 1872 he played a prominent part in the building of the Elizabethtown and Paducah Railroad under Mr. George McLeod. At the close of that assignment he went to Texas as Chief of Party to make the early survey for a part of the Texas and Pacific Railroad from Fort Worth, west, during which the surveyors were much harassed by Indian interference.

Mr. Morris then opened and operated the Jefferson Coal Mines, at Jefferson, Ala., but remained there only a few years before returning to railroad construction work, including bridges, tunnels, etc. Later, he became Chief Engineer for the old Louisville, New Albany and Corrydon Railroad (now the Monon Division of the Chicago, Indianapolis and Louisville Railroad). This position he resigned to enter into partnership with Mr. John Serpelle, as contractors on railroad construction work.

About 1888 he located the rock asphalt deposit in Breckinridge County, Kentucky, and organized the Breckinridge Asphalt Company, serving as its President. This Company laid many streets in the larger cities, Louisville, Buffalo, N. Y., etc., placing more than twelve miles of pavement in Buffalo alone.

In 1908 Mr. Morris was appointed Building Inspector for the City of Louisville, drafting the Building Code which was put into effect in 1909. From this time he was retired from active field work, doing only such consulting work as his then failing eyesight would permit.

In 1876 he was married to Tenie Witherspoon, of Longview, Tex., who died in 1904.

His death occurred on September 29, 1925, at the residence of his daughter, Mrs. George A. Haynes, at Bedford, Ind., and he was buried in Cave Hill Cemetery, Louisville, Ky. He is survived by his daughter, Mrs. Haynes, and two sons, Marshall Morris, M. Am. Soc. C. E., of Austin, Tex., and John Ford Morris, of Louisville.

Mr. Morris was a member of the Engineers and Architects Club of Louisville. He was also a Charter Member of the Louisville Club, acting as its first Secretary, and bequeathing to it his valuable collection of the *Transactions* of the Society.

Mr. Morris was elected a Member of the American Society of Civil Engineers on March 5, 1873.

JOHN THOMSON, M. Am. Soc. C. E.*

Died March 31, 1926.

John Thomson was born on October 25, 1853, in Fochabers, Scotland, the eldest son of Alexander and Elizabeth Hay Thomson, and was brought to America when a child.

He was educated in the common schools and under special instructors, with mathematics receiving marked attention. At first, a watchmaker, Mr. Thomson later became an engineer and inventor.

For about twenty-five years he was associated with the Colt Company, of Hartford, Conn., for which he invented and designed the printing, embossing and cutting, and creasing presses known to the trade as Colt's Armory Presses. Later, he designed other printing presses for his own Company, the John Thomson Press Company.

In the field of water meters he also became distinguished as Inventor and Manufacturer. The first of such inventions was the Disc Water Meter, made by the Thomson Meter Company, which he established. He later designed the Trident, Disc and Crest Water Meters, and organized for their manufacture the Neptune Meter Company, of Long Island City, N. Y., which is believed to be the largest maker of water meters in the United States.

Almost the last work on which Mr. Thomson was engaged was the design of another water meter known as the Zenith. He also invented a number of features in connection with electric furnaces which are now being applied in industry.

In 1877 he was married to Alice Elizabeth McKee, of Canandaigua, N. Y. She and their two sons and one daughter survive him.

He died in Brooklyn, N. Y., on May 31, 1926. A fine portrait of him is in the Engineers' Club, of New York, with those of other Past-Presidents.

Mr. Thomson was a member of the American Society of Mechanical Engineers, the American Institute of Mining and Metallurgical Engineers, the American Electro-Chemical Society, and the Franklin Institute. He was one of the early members, and for three years President, of the Engineers' Club of New York, and was a member of the Union League Club of New York; the Pilgrims Society; and the Royal Thames Yacht Club and the American Luncheon Club, of London, England.

Mr. Thomson endeared himself to his fellow members of the Society by those qualities which have come to be associated with the highest type of Scotch character—ruggedness of purpose, stern integrity, and active hatred of cant and insincerity in any form. This heritage was a pronounced factor in his life, and his love for Scottish things found expression in his activities as a member of the St. Andrew's Society and the Burns Society, the latter of which he served as President.

* Memoir prepared by Edward E. Bartlett, Esq., New York, N. Y.

Mr. Thomson was elected a member of the American Society of Civil Engineers on March 2, 1887. He served as Director of the Society from 1892 to 1894 and as Treasurer from 1895 to 1899.

ANSON WOOD BURCHARD, Assoc. M. Am. Soc. C. E.*

Died January 22, 1927.

In the death of Anson Wood Burchard, on January 22, 1927, the engineering industries of the United States lost a man of notable creative accomplishments.

Born at Hoosick Falls, N. Y., on April 21, 1865, Mr. Burchard received his earlier education in the public schools and then attended Stevens Institute of Technology, at Hoboken, N. J., from which he was graduated in 1885 with the degree of Mechanical Engineer. He was also the recipient in 1926 of the Honorary Degree of Doctor of Laws from Union College.

Mr. Burchard's early employment was along the line of his course at Stevens Institute of Technology, which prepares more particularly for mechanical work. His first fifteen years of experience were in factory engineering and in manufacturing establishments at Danbury, Conn.

Two years (1900 to 1902) of experience as Vice-President of the Cananea Consolidated Copper Company and of the Greene Consolidated Copper Company led to his entering the employ of the General Electric Company, of which he became Comptroller, with headquarters at Schenectady, N. Y. His creative work in conjunction with the President of the General Electric Company, in the successful accomplishment (1902 to 1905) of a consolidation of the Fort Wayne Electric Company, the Stanley Electric Manufacturing Company, the General Incandescent Arc Light Company, the Northern Electric Company, and the Sprague Electric Company as units of the General Electric Company was an achievement of the highest order. He thus was instrumental in aiding to develop a great manufacturing organization with financial strength sufficient to enable it to carry out designs and spend money in research far beyond the resources of any of its individual component companies.

His career is another illustration of the manner in which graduates from engineering schools have capably grasped the problems of finance and administration in the engineering industries. Mr. Burchard's advancement with the General Electric Company was through the post of Assistant to the President, Vice-President, and member of the Board of Directors. In 1922 he was elected Vice-Chairman of the Board of Directors and Chairman of the Executive Committee of the General Electric Company, and, in the same year, President and Chairman of the Board of the International General Electric Company. The Presidency of the latter Company he relinquished subsequently, but continued as Chairman of the Board.

* Memoir prepared by H. de B. Parsons and Dugald C. Jackson, Members, Am. Soc. C. E.

During the World War, Mr. Burchard volunteered as Assistant to the Secretary of War and served from January, 1918, until March, 1919, giving particular attention to the construction and development in the United States of increased facilities for the production of munitions of war.

His personal characteristics and wise judgment in finance and administration, as related to the engineering industries, led to his becoming a constantly sought counsellor in the development of public utility facilities both at home and abroad. He had a master mind for formulating policies in developing the financial resources and credit of public utilities, thus enabling them to increase their buying power, and for expanding the importance of central station work, which resulted in stimulating the electric industry throughout the world.

Mr. Burchard's sound judgment led to his selection as a Director or Trustee of many organizations, namely, American Gas and Electric Company; American Power and Light Company; Andersen, Meyer and Company; Asheville Power and Light Company; American and Foreign Power Company, Incorporated; Barcelona Traction, Light and Power Company; California Electric Generating Company; Carolina Power and Light Company; Central States Electric Corporation; Electric Investment Corporation; Electric Investors Incorporated; Electric Power and Light Corporation; Electric Railway Equipment Securities Corporation; Electrical Securities Corporation; General Electric Company; International General Electric Company; International Power Securities Corporation; Lehigh Power Securities Corporation; Mercantile Safe Deposit Company; Montana Power Company; Mohawk-Hudson Power Corporation; North American Company; New England Power Association; Northeastern Power Corporation; Power Securities Corporation; Republic Railway and Light Company; United States and Foreign Power Securities Corporation; Utah Securities Corporation; United Electric Securities Corporation; Western Power Corporation; Worthington Pump and Machinery Corporation; Yadkin River Power Corporation; The British Thomson-Houston Company, Limited; Compagnie des Lampes; Compagnie Française Thomson Houston; Compagnie Generale di Elettricita; Société d' Électricité et de Mécanique; Sociedad Iberica de Construcciones Electricas; China General Edison Company, Incorporated; Société Edison Clerici Fabbrica Lampade; Société Financiere pour le Développement de l' Électricité; and Nippon Denki Shoken Kabushiku Kaisha (Japanese Electric Bond and Share Company).

Mr. Burchard differed from many other men because as he became older and more stressed by business details, he took upon himself outside personal work. As years passed, he undertook to help others in an increasing degree. He made time for interviews and was always helpful in giving assistance, to the end that the outsider's problem became his problem. This made him admired by all, and he was personally liked not only by his friends, but by his employees.

He had a knowledge of and confidence in fundamentals, whether of scientific, financial, or business character, and the result was clear thinking and correct application of efforts to achieve an end. Similarly, he was possessed of that peculiar faculty of intuition whereby he could reach into a subject-

matter and at once see the facts in their correct relationship. It was due to this faculty that much of his success was founded.

He also had the ability to handle men having divergent views, and to get them together in agreement and accord. His interest in the development of science as applied in manufacturing, and his interest in education, are indicated by his membership in the Charles A. Coffin Foundation Committee and his Trusteeship of Stevens Institute of Technology.

Mr. Burchard was a member of various engineering societies, including the Iron and Steel Institute of Great Britain, the American Society of Mechanical Engineers, and the American Institute of Electrical Engineers.

His sociability and regard for his fellow man are indicated by the list of his clubs, which included the Metropolitan, of Washington, D. C., and Metropolitan, University, Engineers', Racquet and Tennis, Automobile, Westchester Country, Recess, City-Midday, Bankers, Piping Rock, and Nassau Country, in the New York Metropolitan District.

He was married in London, England, on December 5, 1912, to Allene (Tew) Hostetter, daughter of Charles H. Tew, a retired banker of Jamestown, N. Y.

His death occurred suddenly of acute indigestion, and came as a shock to all his associates. Tributes expressed by his friends and colleagues came from over the world. A few sentences from Mr. Owen D. Young give their general consensus:

"Due to his long association with the Company [The General Electric Company], Mr. Burchard had become familiar with its business in all its branches. His loyalty to the Company and his pride in it, his devotion to his associates, and his wide interest in all good causes, made him universally respected and loved. His loss to all of us is deeply felt."

Mr. Burchard was elected an Associate Member of the American Society of Civil Engineers on May 3, 1893.

DAHYABHAI BALABHAI KORA, Assoc. M. Am. Soc. C. E.*

Died September 22, 1926.

Dahyabhai Balabhai Kora was born of a noble family of Jain Baniss, in Kaira, now Jamnagar, Bombay, India, on September 26, 1886. After receiving his early education in Gondal, India, he was graduated in 1908 from the Poona College where he received scholarships every year. He received the degree of Licensed Civil Engineer from the University of Bombay.

In 1909 and 1910, he served with the Public Works Department of the State of Baroda, India, on building construction, roads, sanitation, drainage, etc.

From 1910 to 1917, Mr. Kora was engaged as Assistant Engineer of the State of Gondal, India, on the design and construction of buildings, roads, bridges, irrigation, water-works, town improvement, gas and telephone works. During this period he designed and constructed 50 miles of district roads, including highway, arch, and skew bridges. He also made a survey and design

* Memoir compiled from information on file at the Headquarters of the Society.

for a causeway across the River Vinu and for a drainage project for a part of the City of Gondal. His work also included an impounding reservoir for a combined irrigation and water supply works and the preliminary survey and design for an irrigation plan at Gomta, as well as an irrigation project with an impounding reservoir of earthen dams and masonry waste weir to irrigate about 3 000 acres. In addition, Mr. Kora designed and constructed many public buildings for the State of Gondal, including schools, museums, public offices, etc., using the purely Hindu style of architecture.

From 1911 to 1918, in addition to other duties, he acted as Honorary Examiner in Architecture and Civil Engineering at the Technical Institute, State of Baroda, except for two years when he was Officer in Charge of the Gondal State Camp, Delhi Durbar.

In 1917, Mr. Kora was engaged as Temporary Engineer for the Government of the Central Provinces, India, in charge of the Chhindwara District, on the maintenance and repair of 200 miles of district roads, the construction of a new road, 12 miles long, bonded warehouses, bridges, etc.

In 1918, he was appointed Engineer for Irrigation and Communications, which position he held until September, 1918, when he was made State Engineer of Nawanagar, India, in charge of the Public Works Department and Irrigation. As such, he was engaged on the design and construction of important public buildings, roads, bridges, harbor, irrigation, sanitary, and reclamation works, costing in all more than £1 000 000 sterling. He constructed twelve masonry dams for irrigation across catchment areas from 5 to 200 sq. miles in size and two bunds to reclaim about 5 000 acres of land, as well as approximately 60 miles of district roads and two raised causeways across a river having a catchment basin of 125 sq. miles. Many of the fine public and State buildings by which the City of Jamnagar has attained the reputation of being the most beautiful in Kathiawar, were designed and erected under Mr. Kora's careful supervision.

Mr. Kora was an able engineer and a skillfull designer who carried out his work with a high sense of duty, honesty, and straightforwardness. In his private as well as in his public life he endeared himself to all by his genial disposition, scrupulous honesty, and conscientious discharge of his various duties. His character was exemplary and his manners were simple and unassuming.

The State and the public expressed their deep sorrow at Mr. Kora's death by closing all offices, schools, and shops, in his honor. An Extraordinary Gazette of the State was also published, eulogizing the numerous services rendered by him, and public meetings were held at various places expressing deep sympathy with his parents and family.

In spite of his arduous responsibilities he kept in touch with the progress of engineering science in foreign countries. He was a member of various technical societies, including the Society of Engineers; Institute of Structural Engineers; Institute of Municipal and County Engineers; Royal Sanitary Institute; and the Institute of Indian Engineers. He was also associated with the American Institute of Architects and the Royal Institute of British

Architects, and had contributed important papers at various engineering conferences.

Mr. Kora was elected an Associate Member of the American Society of Civil Engineers on August 9, 1920.

DAVID THOMAS PITKETHLY, Assoc. M. Am. Soc. C. E.*

Died April 21, 1926.

David Thomas Pitkethly was born in Boston, Mass., on March 29, 1876.

In the fall of 1890 he entered the employ of Wheeler and Parks, Consulting Engineers, of Boston, and served as office boy and Assistant on drafting and surveying until 1894. In 1895, he was with George W. Stadley and Company, Map Publishers, Boston, on surveys for atlases, and, in 1896 and 1897, he held the position of Draftsman in the Sewer Department of Boston. In 1898 he was with the Central Railroad of New Jersey on railroad locations, and in 1899 and 1900, was employed by Albert L. Webster, M. Am. Soc. C. E., Sanitary Engineer, of New York, N. Y., on sewage disposal and sanitary engineering work.

From 1900 to 1903, Mr. Pitkethly was engaged as a Draftsman with the Tax Department, City of New York, and from 1904 to August, 1921, served with the Bureau of Sewers, Borough of Brooklyn, as Draftsman and Assistant Engineer. He collaborated on the design of some of the largest sewers built in the Borough and was in charge of a group of engineers and draftsmen who developed the drainage plans and prepared the contract plans for work costing many millions of dollars. He was an exceptionally rapid worker and was brilliant in his engineering conceptions.

In 1913 Mr. Pitkethly became so ill that his life was despaired of; but after a year he returned to his work. His health failed again in 1921 and he was transferred in August to the Board of Water Supply in the Reservoir Division at Grand Gorge, N. Y. During his stay in Grand Gorge he was connected with the development of work on the Schoharie water-shed as part of the Catskill Mountain Water Supply System, except during part of 1922 when he served as Section Engineer in the Aqueduct Department of the Board of Water Supply on the installation of some additional pipes in the steel-pipe siphons of the Catskill Aqueduct.

Mr. Pitkethly was taken seriously ill after returning, in February, 1926, to Roxbury, N. Y., from the dinner given in New York to the Chief Engineer of the Board of Water Supply. He died at his home in Roxbury, on April 21, 1926, and was buried in Cypress Hills Cemetery, in Brooklyn.

His widow, Josephine A. (Burrucker) Pitkethly, and his two children, Edith Jane and David A. Pitkethly, survive him.

Mr. Pitkethly was elected an Associate Member of the American Society of Civil Engineers on April 6, 1909.

* Memoir prepared by John Charles Riedel, M. Am. Soc. C. E.

CHARLES ENGLISH SHEARER, Assoc. M. Am. Soc. C. E.*

Died May 8, 1925.

Charles English Shearer, the son of William Marion and Margaret (English) Shearer, was born in Indianapolis, Ind., on May 17, 1881. He was educated in the public schools of Indianapolis and, later, entered Purdue University, from which he was graduated in 1905 with the degree of Bachelor of Science in Civil Engineering.

After graduation, Mr. Shearer entered the Bridge Department of the Delaware, Lackawanna and Western Railroad Company, at Hoboken, N. J., where he was engaged in detailing and designing highway bridge crossings and miscellaneous work on elimination of grade crossings in New Jersey.

In 1906, he accepted a position as Checker, with the McClintic-Marshall Construction Company, at the Rankin Plant, near Pittsburgh, Pa. From 1906 to 1910 he was connected with the Memphis Bridge Company, of which he became Chief Engineer in 1907 in charge of engineering designs, including erection of highway bridges, towers, tanks, and structural steel buildings.

In 1910 Mr. Shearer opened an office in Memphis, Tenn., as Structural and Industrial Engineer, in which work he spent the next few years. He supervised the design and construction of the Loosa Hatchie River Bridge and, in 1911, was engaged in the design and erection of water towers, steel stacks, steel barges, and levee building machines. From 1912 to 1915 he designed and superintended additions to the power plant of the Memphis Street Railway Company, at Memphis, and, as Associate Engineer, also designed a twenty-story office building. Later, in 1916, he had charge of the design and erection of a retort building for the Forest Product Chemical Company, at Memphis.

In 1918 and 1919, Mr. Shearer was Shop Inspector of steel for the United States Government, Ordnance Department Contract T-2, Nitrate Plant No. 1, at Sheffield, Ala., and was especially appointed by the Bureau of Public Works to the Bureau of Insular Affairs, U. S. Government, Ordnance Department, at Manila, Philippine Islands, to design and detail the structural work of the $750 000 Jones Bridge.

On his return, he again took up structural work in the South. During 1920 he reported on, designed, detailed, and supervised the construction of the Second Street Bridge at Clarksdale, Miss., and, in 1922, he was Structural Engineer on the Auditorium Market House, at Memphis. In 1923 he supervised the structural and mechanical design for flood-gates for the Farelly Lake Levee District, Jefferson County, Arkansas. Later, in 1924, he had charge of the re-design and construction of the Submergible and Lift Bridge, at Wisner, La. He also designed and supervised the construction of the Memphis Power and Light Company's Distribution Building at Memphis.

Mr. Shearer had, by his technical ability, personal magnetism, and strict attention to business, reached a stage in his profession, where his services were constantly sought.

* Memoir prepared by J. H. Haylow, M. Am. Soc. C. E.

He lost his life, when the U. S. Steamer *Norman* sank in the Mississippi River, about ten miles south of Memphis, carrying twenty-three of the passengers and crew to their death. He was attending the First Convention of the Engineers of the Mid-South. A meeting was being held at the time in the cabin of the steamer, by local members of the Society, and those resident in the vicinity, for the purpose of forming a Local Section of the Society.

On August 31, 1911, he was married to Annette Ostrander, who, with three minor children, two sons and a daughter, survives him.

Mr. Shearer was elected an Associate Member of the American Society of Civil Engineers on December 6, 1910. He was also a member of the Engineers Club of Memphis.

PEYTON BROWN WINFREE, Assoc. M. Am. Soc. C. E.*

Died May 9, 1924.

Peyton Brown Winfree was born at Lynchburg, Va., on September 10, 1868, the son of Major Christopher V. and Virginia A. (Brown) Winfree. All his early life, as well as his later years, was spent in his native city. He was educated in the primary and secondary schools of Lynchburg, completing his academic education at Randolph-Macon College, from which he was graduated as Bachelor of Arts in 1888. He then entered Lehigh University where he earned the degree of Civil Engineer in 1891. Later, he took graduate work leading to the degree of Mining Engineer.

His practical work as an engineer was begun in Pennsylvania, where he was successively engaged in locating pipe lines from Athens to Wilkes-Barre, making preliminary surveys for suburban street railways, and in the location of oil properties. In May, 1894, Mr. Winfree became City Engineer of Bradford, Pa., which position he filled with great credit. The death of his father in June, 1902, compelled his return to Lynchburg, where, notwithstanding the great amount of work necessitated by the administering of large affairs connected with his father's estate, he still had time to devote to his chosen profession.

During 1902 and 1903, he was Consulting Engineer for several coal companies in West Virginia. In 1904, he undertook the location of a pipe line through the mountains for conveying water to Lynchburg, and his skill was such that the city was saved thousands of dollars in its construction. His acumen was so great that he was finally given charge of the construction of the entire water system. In this work he was associated with H. L. Shaner, M. Am. Soc. C. E., Chief Engineer, and James H. Fuertes, M. Am. Soc. C. E., as Consulting Engineer. After the completion of the water supply system at Lynchburg Mr. Winfree undertook a contract for the Atlanta, Birmingham, and Atlantic Railway Company, in Alabama. About 1909, he became Vice-President and General Manager of the Glamorgan Pipe and Foundry Company, where his skill as an engineer was constantly utilized. This position he occupied until the time of his death on May 9, 1924. There was universal sorrow at his death on the part of every young engineer who had been employed

* Memoir prepared by C. L. DeMott, Assoc. M. Am. Soc. C. E.

under him. He always showed a breadth of understanding and sympathy that completely won their confidence and loyalty.

In 1911, he was appointed Major of Engineers and was assigned to the staff of General C. C. Vaughan, commanding the First Brigade of Virginia Volunteers. His engineering skill was shown in the selection and construction of camps, of which a notable instance is the camp and rifle range at Cape Henry. He resigned his commission on the re-organization of the State Militia into the National Guard.

The Engineering Profession in the State of Virginia will probably remember Major Winfree longest for his indefatigable work in having enacted into law the requirements for registration of all engineers. His shrewd common sense, pleasing personality, and intimate knowledge of the need for such a law, coupled with his financial independence, combined to make him incomparable as its promoter. From 1920, when the matter was first brought to the attention of the State Legislature, until March, 1924, when there was finally enacted a workable statute, his influence and means were unsparingly used. Governor Davis honored him by an appointment on the original Board created by this statute for the Registration of Engineers and Architects. He was re-appointed by Governor Trinkle and served until his death.

Peyton Winfree was a fine exemplification of the very best of manhood. His services were always at the disposition of his profession, but were by no means restricted to it, as they were freely given to his neighbors, his city, his country, and his Church.

On the change of the form of the government of the City of Lynchburg in 1920, he was elected as one of the five members of the City Governing Council, and was re-nominated to succeed himself at the spring primary, held just prior to his death.

Major Winfree was one of the District Executives who were engaged in compiling and classifying statistics of the economic resources and industrial activities of the United States, under the direction of the Naval Consulting Board, just prior to the entry of the United States into the World War.

It is given to few men to know of the spiritual life of another man, but surely Peyton Winfree belonged to the Abou Ben Adhem class. No man could have been the open-handed, unostentatious neighbor that he was without having a great deal of the love of God in his make-up. Outwardly, not conspicuously religious, those not connected with his church knew that he was constant in his attendance at religious worship, and he was for years one of the Stewards of the Memorial Methodist Episcopal Church of Lynchburg.

He was a member of the Beta Theta Pi Fraternity, of the Benevolent and Protective Order of Elks, and a Free Mason.

On November 25, 1896, he was married to Miss Mabel Wilbur, of Bradford, Pa. By this union he had three sons and a daughter, Christopher V., Wilbur W., Peyton B., Jr., and Louise Winfree.

Mr. Winfree was elected a Junior of the American Society of Civil Engineers on October 2, 1894, and an Associate Member on May 3, 1898.